MENSCH, THE REINFORCED CONCRETE POCKET BOOK

J McMillan
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THE
REINFORCED CONCRETE
POCKET BOOK

Containing Useful Tables, Rules and Illustrations for the Convenient Design, Rational Construction and Ready Computation of Cost of Reinforced Concrete GIRDERS, SLABS, FOOTINGS, COLUMNS, BUILDINGS, RETAINING WALLS, TANKS, GRAIN ELEVATORS, COAL BINS, WATER PIPES, SEWERS, DAMS, BRIDGES, SMOKE STACKS, PILES, ETC., ETC. : : :

BY

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Price, $10.00.

PREFACE

The object of this pocket book is to place before the public in as concise a form as possible practical information in regard to reinforced concrete construction, and the author hopes that this book will promote the use of reinforced concrete and be the medium of its standardization.

It is no textbook, and a good knowledge of mechanics and building construction is indispensible for its rational use, and those thinking that they may become experts by studying this book are warned herewith to be careful and to consult with experienced men before embarking on any important piece of work.

The tables were very carefully figured and checked repeatedly by slide-rule; most of them were figured seven to ten years ago and have been thoroughly tested by a great many practical applications. No claim is made that the book contains anything entirely new except the shape in which the information is presented; and nothing new has been discovered since the tables were first prepared which warranted the author revising them.

A factor of safety of four was universally adopted, and where no other mention is made the concrete mixture was assumed what is commonly called a 1:2:4 mixture=one bag of cement to six cubic feet of aggregate.

The use of high carbon steel is of advantage in reinforced concrete construction. The gain over mild steel is, however, but little for low percentages of reinforcement, and as most of the structures shown in this book are reinforced by only ¼ to ½%, the substitution of mild steel will not decrease the strength of the structures more than 10 to 15%.

The compact form of the book compelled the author to confine the information given to those subjects which cannot readily be found in other books and to confine himself only to those applications of reinforced concrete mostly used.

PROPERTIES OF SQUARE RODS.

Thickness in inches	Area in Sq. Ins. of One Rod	Weight per foot of One Rod	Area in Sq. Ins. of Four Rods	Weight per foot in lbs. of 4 Rods incl. Stirrups and Overlaps	Mixed Rods Two of one kind " " other kind	Area in Sq. Ins.	Weight per foot in lbs. of 4 mixed Rods incl. Stirr. & Overlaps
$\frac{1}{4}$	.062	.212					
$\frac{5}{16}$	.098	.333					
$\frac{3}{8}$	.141	.478					
$\frac{7}{16}$	.191	.651					
$\frac{1}{2}$	.250	.850	1.00	4.5	2–$\frac{1}{2}$ & 2–$\frac{5}{8}$	1.28	5.8
$\frac{5}{8}$	.391	1.33	1.56	7.0	2–$\frac{5}{8}$ & 2–$\frac{3}{4}$	1.90	8.6
$\frac{3}{4}$	.562	1.91	2.25	10.0	2–$\frac{3}{4}$ & 2–$\frac{7}{8}$	2.65	12.0
$\frac{7}{8}$	.766	2.60	3.06	13.8	2–$\frac{7}{8}$ & 2–1″	3.53	15.8
1	1.00	3.40	4.00	18.0	2–1″ & 2–$1\frac{1}{8}$	4.50	20.3
$1\frac{1}{8}$	1.26	4.30	5.04	22.5	2–$1\frac{1}{8}$ & 2–$1\frac{1}{4}$	5.65	25.5
$1\frac{1}{4}$	1.56	5.31	6.25	28.0	2–$1\frac{1}{4}$ & 2–$1\frac{3}{8}$	6.90	31.0
$1\frac{3}{8}$	1.89	6.43	7.56	34.0	2–$1\frac{3}{8}$ & 2–$1\frac{1}{2}$	8.28	37.0
$1\frac{1}{2}$	2.25	7.65	9.00	40.5			

SIX RODS.

THICKNESS OF RODS	$\frac{3}{4}$	$\frac{7}{8}$	1″	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
Area	3.37	4.59	6.00	7.56	9.37	11.34	13.5
Weight per foot incl. Overlaps and Stirrups	15.0	20.5	27.0	34.0	42.0	51.0	60.9
Area of Six Rods 3 one kind, 3 other kind	3.98	5.29	6.79	8.48	10.35	12.42	
Weight per foot incl. Stirrups and Overlaps	18.0	23.7	30.5	38.0	46.5	56.0	

EIGHT RODS.

THICKNESS OF RODS	$\frac{3}{4}$	$\frac{7}{8}$	1″	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
Area	4.5	6.12	8.0	10.08	12.50	15.12	18.00
Weight incl. Overlaps and Stirrups	20.3	27.5	36.0	45.0	56.0	68.0	81.0

The theoretical weight per lineal foot of reinforcing is found by multiplying the area by 3.4. In beams and girders, the rods extend beyond the center of supports, and on this

PROPERTIES OF ROUND RODS.

Diameter Inches	Area in Sq. Ins. of One Rod	Weight per foot of One Rod	Area in Sq. Ins. of Four Rods	Weight per foot of Four Rods incl. Stirrups and Overlaps	Mixed Rods Two of one kind " " other kind	Area in Sq. Ins. of Four mixed Rods	Weight per foot of Four mixed Rods inclusive Stirrups & Overlaps
$\frac{1}{4}$	.049	.167					
$\frac{5}{16}$	.077	.261					
$\frac{3}{8}$	.110	.375					
$\frac{7}{16}$	.150	.511					
$\frac{1}{2}$	.196	.667	.785	3.7	2–$\frac{1}{2}$ & 2–$\frac{5}{8}$	1.00	4.5
$\frac{5}{8}$	.306	1.04	1.227	5.4	2–$\frac{5}{8}$ & 2–$\frac{3}{4}$	1.49	6.7
$\frac{3}{4}$	.442	1.50	1.76	7.9	2–$\frac{3}{4}$ & 2–$\frac{7}{8}$	2.08	9.5
$\frac{7}{8}$	.601	2.04	2.40	10.8	2–$\frac{7}{8}$ & 2–1″	2.77	12.4
1″	.785	2.67	3.14	14.1	2–1″ & 2–1$\frac{1}{8}$	3.55	16.0
1$\frac{1}{8}$	.992	3.38	3.97	17.8	2–1$\frac{1}{8}$ & 2–1$\frac{1}{4}$	4.43	20.0
1$\frac{1}{4}$	1.225	4.17	4.90	22.0	2–1$\frac{1}{4}$ & 2–1$\frac{3}{8}$	5.42	24.5
1$\frac{3}{8}$	1.485	5.05	5.94	26.7	2–1$\frac{3}{8}$ & 2–1$\frac{1}{2}$	6.50	29.3
1$\frac{1}{2}$	1.765	6.01	7.06	31.7			

SIX RODS.

DIAMETER	$\frac{3}{4}$	$\frac{7}{8}$	1″	1$\frac{1}{8}$	1$\frac{1}{4}$	1$\frac{3}{8}$	1$\frac{1}{2}$
Area	2.65	3.60	4.71	5.96	7.36	8.90	10.60
Weight per foot incl. Overlaps & Stirrups	12.0	16.0	21.2	27.0	33.0	40.0	48.0
Area of Six Rods 3 one kind, 3 other kind	3.13	4.15	5.35	6.6	8.0	9.7	
Weight per foot incl. Stirrups and Overlaps	14.1	18.7	24.0	29.7	36.0	43.6	

EIGHT RODS.

DIAMETER	$\frac{3}{4}$	$\frac{7}{8}$	1″	1$\frac{1}{8}$	1$\frac{1}{4}$	1$\frac{3}{8}$	1$\frac{1}{2}$
Area	3.52	4.80	6.28	7.94	9.80	11.88	14.12
Weight incl. Stirrups & Overlaps	15.8	21.6	28.2	35.7	44.0	53.5	63.5

account and on account of the stirrups the coefficient of 3.4 increases to an average of 4.5. The stirrups alone weigh from 5 to 10% of the weight of the rods.

PROPERTIES OF TEE BEAMS.

ROUND BARS						SQUARE BARS	
Number of Beam	Bending Moment in 1000 Foot-lbs.	Width and Depth of Beam Below Slab Inches Shear at 60 Pounds per Square Inch	Reinforcement —— Round or Square	Weight of Steel per Lineal Foot Including Stirrups and Overlaps	Cubic Feet of Concrete per Lineal Foot	Bending Moment in 1000 Foot-Pounds	Weight of Steel per Lineal Foot Including Stirrups and Overlaps
1	4.0	3½x6¾	2–½	1.8	.165	5.0	2.3
2	6.2	[2300]	2–⅝	2.7	"	7.8	3.5
3	9.0	"	2–¾	4.0	"	11.3	5.0
4	8.0	5½x8¾	2–⅝	2.7	.34	10.2	3.5
5	10.3	[4200]	4–½	3.7	"	13.1	4.5
6	11.6	"	2–¾	4.0	"	14.8	5.0
7	16.0	"	4–⅝	5.4	"	20.4	7.0
8	23.1	"	4–¾	7.9	"	29.5	10.0
9	27.4	7½x8¾	2–¾; 2–⅞	9.5	.46	34.7	12.0
10	31.5	[5700]	4–⅞	10.8	"	40.0	13.8
11	19.7	5½x10¾	4–⅝	5.4	.42	25.2	7.0
12	24.0	[4860]	2–⅝; 2¾	6.7	"	30.6	8.6
13	28.4	"	4–¾	7.9	"	36.2	10.0
14	33.5	"	2–¾; 2–⅞	9.5	"	42.7	12.0
15	38.6	7½x10¾	4–⅞	10.8	.56	49.5	13.8
16	44.0	[6620]	2–⅞–2–1″	12.4	"	57.0	15.8
17	50.3	"	4–1″	14.1	"	64.3	18.0
18	57.0	"	2–1; 2–1⅛	16.0	"	72.3	20.3
19	23.3	5½x12¾	4–⅝	5.4	.49	29.8	7.0
20	28.5	[5520]	2–⅝; 2–¾	6.7	"	36.4	8.6
21	33.6	"	4–¾	7.9	"	43.0	10.0
22	40.0	"	2–¾; 2–⅞	9.5	"	50.8	12.0
23	46.0	7½x12¾	4–⅞	10.8	.67	58.5	13.8
24	53.0	[7500]	2–⅞; 2–1″	12.4	"	67.5	15.8
25	60.0	"	4–1″	14.1	"	76.4	18.0
26	67.8	9½x12¾	2–1″; 2–1–⅛	16.0	.84	86.0	20.3
27	76.1	[9500]	4–1⅛	17.8	"	96.5	22.5
28	37.6	7½x14¼	4–¾	7.9	.75	48.0	10.0
29	44.5	[8650]	2–¾; 2–⅞	9.5	"	56.5	12.0
30	51.2	"	4–⅞	10.8	"	65.0	13.8
31	59.0	"	2–⅞; 2–1″	12.4	"	75.0	15.8
32	67.0	9½x14¼	4–1″	14.1	.95	85.0	18.0
33	76.0	[11000]	2–1″; 2–1⅛	16.0	"	96.3	20.3

PROPERTIES OF TEE BEAMS.

ROUND BARS						SQUARE BARS	
Number of Beam	Bending Moment in 1000 Foot-lbs.	Width and Depth of Beam Below Slab Shear at 60 Pounds per Square Inch	Reinforcement —— Round or Square	Weight of Steel per Lineal Foot Including Stirrups and Overlaps	Cubic Feet of Concrete per Lineal Foot	Bending Moment in 1000 Foot-Pounds	Weight of Steel per Lineal Foot Including Stirrups and Overlaps
34	85.0	9½"x14¼"	4–1⅛"	17.8	.95	108.0	22.5
35	94.8	[11000]	2–1⅛; 2–1¼	20.0	.95	121.0	25.5
36	105.0	11½x14¼	4–1¼	22.0	1.15	134.0	28.0
37	127.0	[13300]	6–1⅛	27.0	"	162.0	34.0
38	58.5	7½x16¼	4–⅞	10.8	.85	74.7	13.8
39	67.3	[9550]	2–⅞; 2–1	12.4	.85	86.0	15.8
40	76.5	9½x16¼	4–1"	14.1	1.07	97.5	18.0
41	86.5	[12100]	2–1; 2–1⅛	16.0	"	110.0	20.3
42	97.0	"	4–1⅛	17.8	"	123.0	22.5
43	108.0	"	2–1⅛; 2–1¼	20.0	"	138.0	25.5
44	119.5	11½x16¼	4–1¼	22.0	1.30	152.0	28.0
45	145.0	[14600]	6–1⅛	27.0	"	184.0	34.0
46	86.0	9½x18¼	4–1"	14.1	1.20	110.0	18.0
47	97.2	[13200]	2–1; 2–1⅛	16.0	"	123.0	20.3
48	109.0	"	4–1⅛	17.8	"	138.0	22.5
49	129.0	"	6–1"	21.2	"	164.0	27.0
50	121.0	"	2–1⅛; 2–1¼	20.0	"	155.0	25.5
51	147.0	11½x18¼	3–1; 3–1⅛	24.0	1.45	186.0	30.5
52	134.0	[16000]	4–1¼	22.0	"	172.0	28.0
53	163.0	"	6–1⅛	27.0	"	207.0	34.0
54	181.0	"	3–1⅛; 3–1¼	29.7	"	232.0	38.0
55	135.0	9½x20¼	2–1⅛; 2–1¼	20.0	1.34	172.0	25.5
56	143.0	[14400]	6–1"	21.2	"	182.0	27.0
57	149.0	11½x20¼	4–1¼	22.0	1.62	190.0	28.0
58	163.0	[17400]	3–1; 3–1⅛	24.0	"	206.0	30.5
59	181.0	"	6–1⅛	27.0	"	229.0	34.0
60	201.0	"	3–1⅛; 3–1¼	29.7	"	257.0	38.0
61	178.0	11½x22¼	3–1; 3–1⅛	24.0	1.80	226.0	30.5
62	198.0	[18800]	6–1⅛	27.0	"	253.0	34.0
63	220.0	"	3–1⅛; 3–1¼	29.7	"	283.0	38.0
64	245.0	"	6–1¼	33.0	"	312.0	42.0
65	267.0	"	3–1¼; 3–1⅜	36.0	"	345.0	46.5

The sizes of beams and reinforcement in each horizontal line refer both to round and square rods.

PROPERTIES OF TEE BEAMS.

ROUND BARS						SQUARE BARS	
Number of Beam	Bending Moment in 1000 Foot-lbs.	Width and Depth of Beam Below Slab Shear at 60 Pounds per Square Inch	Reinforcement Round or Square	Weight of Steel per Lineal Foot Including Stirrups and Overlaps	Cubic Feet of Concrete per Lineal Foot	Bending Moment in 1000 Foot-Pounds	Weight of Steel per Lineal Foot Including Stirrups and Overlaps
66	208	11½″x26″	3-1; 3-1⅛	24.0	2.08	265	30.5
67	233		6-1⅛	27.0	"	294	34.0
68	257	[21400]	3-1⅛; 3-1¼	29.7	"	330	38.0
69	287	"	6-1¼	33.0	"	365	42.0
70	312	13½x26	3-1¼; 3-1⅜	36.0	2.44	405	46.5
71	347	[25100]	6-1⅜	40.0	"	445	51.0
72	225	11½x28	3-1; 3-1⅛	24.0	"	285	30.5
73	251	[22700]	6-1⅛	27.0	"	315	34.0
74	277	"	3-1⅛; 3-1¼	29.7	"	356	38.0
75	308	"	6-1¼	33.0	"	394	42.0
76	336	13½x28	3-1¼; 3-1⅜	36.0	2.63	435	46.5
77	374	[26700]	6-1⅜	40.0	"	477	51.0
78	269	11½x30	6-1⅛	27.0	2.40	338	34.0
79	296		3-1⅛; 3-1¼	29.7	"	381	38.0
80	332	[24100]	6-1¼	33.0	"	421	42.0
81	359	"	3-1¼; 3-1⅜	36.0	"	466	46.5
82	400	13½x30	6-1⅜	40.0	2.82	510	51.0
83	440	[28400]	8-1¼	44.0	"	562	56.0
84	529	13½x36	8-1¼″	44.0	3.38	675	56.0
85	640	[33200]	8-1⅜″	53.5	"	816	68.0
86	760	"	8-1½″	63.5	"	972	81.0
87	617	13½x42	8-1¼	44.0	3.95	789	56.0
88	750	[38000]	8-1⅜	53.5	"	951	68.0
89	890	15x42	8-1½	63.5	4.38	1130	81.0
90	703	15x48	8-1¼	44.0	5.00	900	56.0
91	855	[47700]	8-1⅜	53.5	"	1080	68.0
92	1010	"	8-1½	63.5	"	1290	81.0
93	1270	17x48	10-1½	79.5	5.67	1620	101.0
94	1520	[54000]	12-1½	95.5	"	1940	122.0
95	930	15x52	8-1⅜	53.5	5.41	1180	68.0
96	1110	[51100]	8-1½	63.5	"	1400	81.0
97	1380	17x52	10-1½	79.5	6.12	1750	101.0
98	1650	[58000]	12-1½	95.5	"	2120	122.0

PROPERTIES OF TEE BEAMS.

ROUND BARS						SQUARE BARS	
Number of Beam	Bending Moment in 1000 Foot-lbs.	Width and Depth of Beam Below Slab Shear at 60 Pounds per Square Inch	Reinforcement —— Round or Square	Weight of Steel per Lineal Foot Including Stirrups and Overlaps	Cubic Feet of Concrete per Lineal Foot	Bending Moment in 1000 Foot-Pounds	Weight of Steel per Lineal Foot Including Stirrups and Overlaps
201	4.4	4x7½	2-½	1.8	.21	5.6	2.3
202	6.9	[2760]	2-⅝	2.7	"	8.8	3.5
203	9.9	"	2-¾	4.0	"	12.6	5.0
204	8.8	6x9½	2-⅝	2.7	.40	11.1	3.5
205	11.1	"	4-½	3.7	"	14.2	4.5
206	12.7	[4850]	2-¾	4.0	"	16.0	5.0
207	17.4	"	4-⅝	5.4	"	22.2	7.0
208	25.2	"	4-¾	7.9	"	32.0	10.0
209	29.7	8x9½	2-¾; 2-⅞	9.5	.53	37.7	12.0
210	34.2	[6470]	4-⅞	10.8	"	43.5	13.8
211	21.0	6x11½	4-⅝	5.4	.48	26.9	7.0
212	25.6	[5580]	2-⅝; 2-¾	6.7	"	32.8	8.6
213	30.3	"	4-¾	7.9	"	38.8	10.0
214	35.8	"	2-¾; 2-⅞	9.5	"	45.7	12.0
215	41.2	8x11½	4-⅞	10.8	.64	53.0	13.8
216	47.5	"	2-⅞; 2-1"	12.4	"	61.0	15.8
217	54.0	[7430]	4-1"	14.1	"	69.0	18.0
218	61.0	"	2-1"; 2-1⅛	16.0	"	77.5	20.3
219	24.6	6x13½	4-⅝	5.4	.56	31.6	7.0
220	30.1	"	2-⅝; 2-¾	6.7	"	38.5	8.6
221	35.6	[6300]	4-¾	7.9	"	45.5	10.0
222	42.0	"	2-¾; 2-⅞	9.5	"	53.7	12.0
223	48.5	8x13½	4-⅞	10.8	.75	62.0	13.8
224	56.0	[8390]	2-⅞; 2-1"	12.4	"	71.5	15.8
225	63.4	"	4-1"	14.1	"	81.0	18.0
226	71.8	10x13½	2-1"; 2-1⅛	16.0	.94	91.0	20.3
227	80.2	[10500]	4-1⅛	17.8	"	102.0	22.5
228	40.9	8x15½	4-¾	7.9	.86	52.1	10.0
229	48.4	[9840]	2-¾; 2-⅞	9.5	"	61.5	12.0
230	55.8	"	4-⅞	10.8	"	71.0	13.8
231	64.1	"	2-⅞; 2-1"	12.4	"	82.0	15.8
232	73.0	10x15½	4-1"	14.1	1.08	93.0	18.0
233	82.4	[12300]	2-1"; 2-1⅛	16.0	"	104.5	20.3

Use these beams, where the lumber can be obtained in full sizes.

PROPERTIES OF TEE BEAMS.

ROUND BARS						SQUARE BARS	
Number of Beam	Bending Moment in 1000 Foot-lbs.	Width and Depth of Beam Below Slab Shear at 60 Pounds per Square Inch	Reinforcement —— Round or Square	Weight of Steel per Lineal Foot Including Stirrups and Overlaps	Cubic Feet of Concrete per Lineal Foot	Bending Moment in 1000 Foot-Pounds	Weight of Steel per Lineal Foot Including Stirrups and Overlaps
234	92.1	10x15½	4–1⅛	17.8	1.08	117.0	22.5
235	103.0	[12300]	2–1⅛; 2–1¼	20.0	“	131.0	25.5
236	114.0	12x15½	4–1¼	22.0	1.30	145.0	28.0
237	138.0	[14700]	6–1⅛	27.0	“	176.0	34.0
238	63.0	8x17½	4–⅞	10.8	.97	80.1	13.8
239	72.9	[10800]	2–⅞; 2–1″	12.4	“	92.8	15.8
240	82.3	10x17½	4–1″	14.1	1.22	105.0	18.0
241	93.2	[13500]	2–1″; 2–1⅛	16.0	“	118.0	20.3
242	104.0	“	4–1⅛	17.8	“	132.5	22.5
243	116.0	“	2–1⅛; 2–1¼	20.0	“	148.0	25.5
244	129.0	12x17½	4–1¼	22.0	1.46	164.0	28.0
245	156.0	[16200]	6–1⅛	27.0	“	198.0	34.0
246	92.0	10x19½	4–1″	14.1	1.36	117.0	18.0
247	104.0	[14700]	2–1″; 2–1⅛	16.0	“	132.0	20.3
248	116.0	“	4–1⅛	17.8	“	148.0	22.5
249	138.0	“	6–1″	21.2	“	177.0	27.0
250	130.0	“	2–1⅛; 2–1¼	20.0	“	165.0	25.5
251	156.0	12x19½	3–1″; 3–1⅛	24.0	1.63	198.0	30.5
252	144.0	“	4–1¼	22.0	“	183.0	28.0
253	174.0	[17600]	6–1⅛	27.0	“	221.0	34.0
254	193.0	“	3–1⅛; 3–1¼	29.7	“	248.0	38.0
255	143.0	10x21½	2–1⅛; 2–1¼	20.0	1.50	182.0	25.5
256	152.0	[15900]	6–1″	21.2	“	193.0	27.0
257	158.0	12x21½	4–1¼	22.0	1.80	202.0	28.0
258	172.0	“	3–1″; 3–1⅛	24.0	“	218.0	30.5
259	192.0	[19100]	6–1⅛	27.0	“	242.0	34.0
260	213.0	“	3–1⅛; 3–1¼	29.7	“	274.0	38.0
261	188.0	12x23½	3–1″; 3–1⅛	24.0	1.96	240.0	30.5
262	210.0	[20500]	6–1⅛	27.0	“	266.0	34.0
263	233.0	“	3–1⅛; 3–1¼	29.7	“	300.0	38.0
264	259.0	14x23½	6–1¼	33.0	2.30	330.0	42.0
265	282.0	[23900]	3–1¼; 3–1⅜	36.0	“	365.0	46.5

Use these beams, where the lumber can be obtained in full sizes.

PROPERTIES OF STEEL BEAMS.

Depth of I Beam Inches	Weight per Foot Pounds	Bending Moment at 16000 pounds Stress 1000 Ft. Lbs.	Width of Flange Inches
3	5½	2.15	2.33
"	7½	2.55	2.50
4	7½	3.93	2.66
"	10½	4.70	2.86
5	9¾	6.45	3.00
"	14¾	8.10	3.28
6	12¼	9.80	3.33
6	17¼	11.70	3.56
7	15	14.00	3.66
"	20	16.20	3.86
8	18	19.00	4.00
"	25	23.00	4.26
9	21	25.00	4.33
"	35	33.40	4.76
10	25	33.00	4.66
"	40	46.80	5.15
12	31½	48.50	5.00
"	40	61.00	5.25
"	55	81.50	5.75
15	42	75.00	5.50
"	60	110.00	6.17
"	80	138.00	6.63
18	55	119.00	6.00
"	70	146.00	6.50
"	90	176.00	7.08
20	65	157.00	6.25
"	80	187.00	6.75
"	100	218.00	7.03
24	80	234.00	7.00
"	90	260.00	7.42
"	100	278.00	7.69

Depth of Channels Inches	Weight per Foot Pounds	Bending Moment at 16000 pounds Stress 1000 Ft. Lbs.	Width of Flange Inches
3	4	1.47	1.41
3	5	1.60	1.51
4	5.25	2.53	1.58
"	6.25	2.80	1.65
5	6.5	4.00	1.75
"	9.0	4.67	1.89
6	8.0	5.73	1.92
"	10.55	6.67	2.04
7	9.75	8.00	2.09
"	12.25	9.20	2.19
8	11.25	10.80	2.26
"	13.75	12.00	2.35
9	13.25	14.00	2.43
"	15.0	15.10	2.47
10	15.0	17.90	2.60
"	20.0	20.90	2.74
12	20.5	28.50	2.94
"	25.0	32.00	3.05
"	30.0	35.90	3.17
"	35.0	39.80	3.29
15	33	55.60	3.40
15	35	57.00	3.44
"	40	61.80	3.53
"	45	66.70	3.63

This table is inserted to facilitate comparisons of steel and reinforced concrete beams.

SECTIONAL AREAS OF ROUND BARS PER LINEAL ALSO THEIR WEIGHTS PER SQUARE

Diameter Inches	Area	SPACING IN INCHES							
	Weight Per Foot	4	4½	5	5½	6	6½	7	7½
1/4	.049 .167	**.149** .790 1.26	**.132** .700 1.11	**.119** .63 1.00	**.108** .57 .91	**.099** .53 .83	**.091** .48 .77	**.085** .45 .72	**.079** .42 .67
5/16	.077 .261	**.230** 1.22 1.93	**.205** 1.09 1.73	**.184** .98 1.55	**.168** .89 1.42	**.154** .82 1.30	**.142** .75 1.20	**.132** .70 1.11	**.123** .65 1.04
3/8	.110 .375	**.332** 1.76 2.80	**.295** 1.56 2.48	**.265** 1.41 2.23	**.242** 1.28 2.04	**.222** 1.18 1.87	**.205** 1.09 1.73	**.190** 1.01 1.60	**.177** .94 1.49
7/16	.150 .511	**.450** 2.38 3.80	**.400** 2.12 3.36	**.362** 1.92 3.05	**.330** 1.75 2.78	**.300** 1.60 2.52	**.278** 1.48 2.34	**.258** 1.37 2.17	**.241** 1.28 2.03
1/2	.196 .667	**.590** 3.13 4.95	**.523** 2.80 4.40	**.471** 2.50 3.96	**.430** 2.28 3.62	**.394** 2.09 3.31	**.363** 1.93 3.05	**.337** 1.79 2.83	**.315** 1.67 2.65
5/8	.306 1.043	**.920** 4.87 7.72	**.820** 4.35 6.90	**.735** 3.90 6.18	**.670** 3.55 5.62	**.613** 3.25 5.16	**.566** 3.00 4.75	**.527** 2.80 4.42	**.490** 2.60 4.12
3/4	.442 1.502	**1.32** 7.00 11.10	**1.18** 6.25 9.92	**1.06** 5.62 8.91	**.965** 5.10 8.10	**.884** 4.68 7.41	**.817** 4.33 6.86	**.758** 4.00 6.36	**.708** 3.75 5.95
7/8	.601 2.044	**1.81** 9.60 15.20	**1.61** 8.5 13.60	**1.45** 7.7 12.20	**1.32** 7.0 11.10	**1.21** 6.4 10.20	**1.11** 5.9 9.30	**1.03** 5.5 8.70	**.964** 5.1 8.10
1	.785 2.667	**2.36** 12.50 19.80	**2.10** 11.1 17.70	**1.89** 10.00 15.90	**1.72** 9.10 14.50	**1.57** 8.30 13.20	**1.45** 7.70 12.20	**1.35** 7.15 11.40	**1.26** 6.70 10.60

The black figures denote the areas. Directly below are the weights in simple, continuous slabs including overlaps and extra rods over supports, except longitudinal reinforcement,

FOOT OF SLAB IF SPACED FROM 4 TO 12″ C. C.; FOOT IN SIMPLE AND SQUARE SLABS.

Diameter Inches	SPACING IN INCHES								
	8	8½	9	9½	10	10½	11	11½	12
1/4	.074 .4 .62	.070 .37 .59	.066 .35 .56	.063 .33 .53	.060 .32 .51	.057 .30 .48	.054 .29 .46	.052 .28 .44	.049 .26 .41
5/16	.115 .61 .97	.108 .57 .91	.102 .54 .86	.097 .52 .82	.092 .49 .77	.088 .47 .74	.084 .45 .71	.080 .43 .67	.077 .41 .65
3/8	.166 .88 1.40	.156 .83 1.31	.148 .79 1.25	.140 .75 1.18	.133 .71 1.12	.126 .67 1.06	.120 .64 1.01	.115 .61 .97	.110 .59 .93
7/16	.226 1.20 1.90	.212 1.12 1.78	.200 1.06 1.68	.190 1.01 1.60	.180 .96 1.52	.172 .91 1.45	.164 .87 1.38	.157 .83 1.32	.150 .80 1.26
1/2	.295 1.57 2.48	.277 1.47 2.33	.262 1.39 2.21	.248 1.32 2.09	.236 1.25 1.98	.225 1.20 1.89	.214 1.14 1.80	.205 1.09 1.73	.196 1.04 1.65
5/8	.460 2.44 3.86	.433 2.30 3.65	.410 2.17 3.45	.387 2.05 3.26	.367 1.95 3.09	.350 1.85 2.95	.335 1.78 2.82	.320 1.70 2.69	.306 1.62 2.57
3/4	.662 3.50 5.57	.623 3.30 5.25	.590 3.13 4.96	.559 2.95 4.70	.530 2.81 4.45	.505 2.68 4.25	.481 2.55 4.05	.460 2.44 3.87	.442 2.35 3.71
7/8	.904 4.8 7.60	.850 4.5 7.15	.800 4.25 6.72	.760 4.03 6.40	.720 3.82 6.05	.685 3.63 5.75	.656 3.48 5.50	.630 3.34 5.30	.601 3.20 5.05
1	1.18 6.25 9.92	1.11 5.90 9.32	1.05 5.60 8.80	.990 5.25 8.32	.940 5.00 7.90	.900 4.76 7.57	.856 4.53 7.20	.820 4.35 6.90	.785 4.15 6.60

which weigh in slabs 3″, 3½″, 4″, 4½″, 5″, 5½″, 6″ thick,
pounds per sq. foot .18, .21, .24, .27, .3, .33, .36
The figures in the third line denote the weights of steel per

SECTIONAL AREAS OF SQUARE BARS PER LINEAL
ALSO THEIR WEIGHTS PER SQUARE

Thickness Inches	Area Weight Per Foot	SPACING IN INCHES							
		4	4½	5	5½	6	6½	7	7½
1/4	.062 .212	.190 1.01 1.60	.167 .88 1.40	.150 .80 1.26	.137 .73 1.15	.125 .66 1.05	.116 .63 .98	.107 .57 .90	.100 .53 .84
5/16	.098 .333	.290 1.54 2.45	.260 1.38 2.18	.233 1.24 1.96	.212 1.12 1.78	.195 1.04 1.64	.180 .96 1.51	.168 .89 1.41	.155 .82 1.30
3/8	.140 .478	.422 2.24 3.55	.375 2.00 3.15	.340 1.80 2.85	.308 1.63 2.60	.281 1.49 2.36	.260 1.38 2.18	.240 1.27 2.02	.225 1.20 1.89
7/16	.191 .651	.574 3.05 4.85	.510 2.70 4.30	.450 2.44 3.80	.418 2.21 3.50	.382 2.02 3.20	.355 1.88 3.00	.327 1.73 2.75	.305 1.62 2.56
1/2	.250 .850	.750 3.97 6.30	.670 3.55 5.62	.600 3.18 5.03	.545 2.88 4.60	.500 2.65 4.20	.462 2.45 3.90	.430 2.28 3.62	.400 2.12 3.36
5/8	.390 1.333	1.172 6.20 9.86	1.04 5.51 8.75	.94 4.98 7.90	.85 4.50 7.13	.781 4.15 6.57	.722 3.82 6.10	.670 3.55 5.62	.625 3.31 5.25
3/4	.562 1.913	1.687 8.95 14.20	1.50 7.95 12.60	1.35 7.15 11.40	1.23 6.51 10.40	1.12 5.95 9.40	1.04 5.51 8.75	.97 5.12 8.14	.900 4.77 7.55
7/8	.766 2.603	2.297 12.20 19.30	2.04 10.80 17.20	1.82 9.66 15.30	1.67 8.85 14.00	1.53 8.10 12.90	1.40 7.43 11.80	1.31 6.95 11.0	1.22 6.48 10.3
1	1.00 3.400	3.000 15.90 25.20	2.67 14.20 22.50	2.40 12.72 20.20	2.18 11.60 18.40	2.00 10.60 16.80	1.85 9.80 15.60	1.72 9.11 14.50	1.60 8.49 13.50

sq. ft. in square slabs including overlaps. The theoretical weight per sq. ft. is obtained by multiplying the areas by 3.4. On account of overlaps and the rods over the sup-

FOOT OF SLAB, IF SPACED FROM 4 TO 12" C. C.; FOOT IN SIMPLE AND SQUARE SLABS.

Thickness Inches	SPACING IN INCHES								
	8	8½	9	9½	10	10½	11	11½	12
1/4	.093 .50 .78	.089 .47 .75	.083 .44 .70	.079 .42 .66	.075 .40 .63	.071 .38 .60	.068 .36 .57	.065 .35 .55	.062 .33 .52
5/16	.145 .77 1.22	.138 .73 1.16	.130 .69 1.10	.124 .66 1.04	.118 .63 .99	.111 .59 .94	.106 .56 .90	.102 .54 .86	.098 .52 .82
3/8	.210 1.12 1.77	.198 1.05 1.67	.188 1.00 1.58	.178 .95 1.50	.170 .90 1.43	.160 .85 1.35	.154 .82 1.30	.146 .78 1.23	.141 .75 1.19
7/16	.286 1.52 2.40	.270 1.43 2.27	.255 1.35 2.15	.242 1.28 2.05	.230 1.22 1.94	.219 1.16 1.84	.209 1.11 1.76	.200 1.06 1.68	.191 1.01 1.61
1/2	.375 1.99 3.15	.353 1.87 2.97	.333 1.76 2.80	.316 1.68 2.65	.300 1.59 2.52	.285 1.51 2.40	.273 1.44 2.30	.261 1.39 2.20	.250 1.33 2.10
5/8	.585 3.10 4.92	.550 2.93 4.62	.520 2.76 4.37	.493 2.61 4.15	.470 2.49 3.95	.445 2.36 3.74	.428 2.26 3.60	.408 2.16 3.42	.391 2.07 3.30
3/4	.850 4.50 7.13	.790 4.18 6.62	.750 3.98 6.30	.710 3.76 5.96	.670 3.55 5.62	.640 3.39 5.38	.613 3.25 5.15	.590 3.13 4.95	.563 2.98 4.73
7/8	1.15 6.10 9.70	1.08 5.72 9.10	1.02 5.41 8.60	.960 5.09 8.10	.910 4.82 7.65	.860 4.56 7.20	.830 4.40 7.0	.795 4.21 6.70	.755 4.06 6.40
1	1.50 7.96 12.60	1.41 7.50 11.80	1.33 7.05 11.20	1.260 6.69 10.60	1.20 6.37 10.10	1.14 6.04 9.60	1.09 5.80 9.20	1.04 5.51 8.80	1.000 5.32 8.40

ports, this coefficient increases to 5.3 in simple continuous slabs and on account of overlaps and rods running in two directions to 8.4 in square slabs.

SAFE BENDING MOMENTS IN FOOT-POUNDS PER PERCENTAGE OF STEEL VARYING FROM

THICKNESS OF SLABS IN INCHES AND FEET									
2″ [.167′]		2½″ [.208′]		3″ [.25′]		3½″ [.292′]		4″ [.333′]	
Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area
200	.06	315	.075	440	.09	600	.105	800	.120
220	.072	340	.090	470	.108	640	.126	850	.144
240	.084	365	.105	500	.126	680	.147	900	.168
260	.096	390	.120	530	.144	720	.168	950	.192
280	.108	415	.135	560	.162	760	.189	1000	.216
300	.120	440	.150	590	.180	800	.210	1050	.240
320	.132	465	.165	620	.198	840	.231	1100	.264
340	.148	490	.180	650	.216	880	.252	1150	.288
360	.160	515	.195	680	.234	920	.273	1200	.312
380	.172	540	.210	710	.252	960	.294	1250	.336
400	.184	565	.225	740	.270	1000	.315	1300	.360
420	.196	590	.240	770	.288	1040	.336	1350	.384
440	.208	615	.255	800	.306	1080	.357	1400	.408
460	.220	640	.270	830	.324	1120	.378	1450	.432
480	.232	665	.285	860	.342	1160	.399	1500	.456
500	.240	690	.300	890	.360	1200	.420	1550	.480
520	.256	715	.315	920	.378	1230	.441	1600	.504
540	.268	740	.330	950	.396	1270	.462	1650	.528
560	.280	765	.345	980	.414	1300	.483	1700	.552
580	.292	790	.360	1010	.432	1335	.504	1750	.576
600	.304	815	.375	1040	.450	1370	.525	1800	.600

The area of steel is for a width of one foot. The spaces denote the percentages of ½, ¾ and 1% of reinforcing. As a universal rule it will be found that the cheapest construc-

LINEAL FOOT OF SLABS, IF REINFORCED BY A ¼ TO 1¼% IN INCREMENTS OF 1/20%.

THICKNESS OF SLABS IN INCHES AND FEET

4½" [.375']		5" [.417']		5½" [.458']		6" [.500']		6½" [.542']	
Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area
1000	.135	1300	.150	1600	.165	1800	.180	2200	.195
1080	.162	1400	.180	1720	.198	1950	.216	2370	.234
1160	.189	1500	.210	1840	.231	2100	.252	2540	.273
1240	.216	1600	.240	1960	.264	2250	.288	2710	.312
1320	.243	1700	.270	2080	.297	2400	.324	2880	.351
1400	.270	1800	.300	2200	.330	2550	.360	3050	.390
1480	.297	1900	.330	2320	.363	2700	.396	3220	.429
1560	.324	2000	.360	2440	.396	2850	.432	3390	.468
1640	.351	2100	.390	2560	.429	3000	.468	3560	.507
1720	.378	2200	.420	2680	.462	3150	.504	3730	.546
1800	.405	2300	.450	2800	.495	3300	.540	3900	.585
1860	.432	2400	.480	2920	.528	3450	.576	4070	.624
1940	.459	2500	.510	3040	.561	3600	.612	4240	.663
2020	.486	2600	.540	3160	.594	3750	.648	4410	.702
2100	.513	2700	.570	3280	.627	3900	.684	4580	.741
2180	.540	2800	.600	3400	.660	4050	.720	4750	.780
2260	.567	2900	.630	3520	.693	4200	.756	4920	.819
2340	.594	3000	.660	3640	.726	4350	.792	5090	.858
2420	.621	3100	.690	3760	.759	4500	.828	5260	.897
2500	.648	3200	.720	3880	.792	4650	.864	5430	.936
2580	.675	3300	.750	4000	.825	4800	.900	5600	.975

tion is obtained, if the percentage of reinforcement is ¼ to ½%. For the proper steel rods, spacing and weights, see preceding pages.

SAFE BENDING MOMENTS IN FOOT-POUNDS PERCENTAGES OF STEEL VARYING FROM

THICKNESS OF SLABS IN INCHES AND FEET

7" [.583']		7½" [.625']		8" [.667']		9" [.750']		10" [.833']	
Bending Moment Foot Lbs.	Sectional Area of Reinforcement	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area
2500	.210	2800	.225	3200	.240	4200	.270	5000	.30
2700	.252	3030	.270	3460	.288	4520	.324	5400	.36
2900	.294	3260	.315	3720	.336	4840	.378	5800	.42
3100	.336	3490	.360	3980	.384	5160	.432	6200	.48
3300	.378	3720	.405	4240	.432	5480	.486	6600	.54
3500	.420	3950	.450	4500	.480	5800	.540	7000	.60
3700	.462	4180	.495	4760	.528	6120	.594	7400	.66
3900	.504	4410	.540	5020	.576	6440	.648	7800	.72
4100	.546	4640	.585	5280	.624	6760	.702	8200	.78
4300	.588	4870	.630	5540	.672	7080	.756	8600	.84
4500	.630	5100	.675	5800	.720	7400	.810	9000	.90
4700	.672	5330	.720	6060	.768	7720	.864	9400	.96
4900	.714	5560	.765	6320	.816	8040	.918	9800	1.02
5100	.756	5790	.810	6580	.864	8360	.972	10200	1.08
5300	.798	6020	.855	6840	.912	8680	1.026	10600	1.14
5500	.846	6250	.900	7100	.960	9000	1.080	11000	1.20
5700	.882	6480	.945	7360	1.008	9320	1.134	11400	1.26
5900	.924	6710	.990	7620	1.056	9640	1.188	11800	1.32
6100	.966	6940	1.035	7880	1.104	9960	1.242	12200	1.38
6300	1.008	7170	1.080	8140	1.152	10280	1.296	12600	1.44
6500	1.050	7400	1.125	8400	1.200	10600	1.350	13000	1.50

The area of steel is for a width of one foot. The spaces denote the percentages of ½, ¾ and 1% of reinforcing. The weight of the steel per square foot is found in case of square slabs by multiplying the area of steel by 8.4; for

PER LINEAL FOOT OF SLABS REINFORCED BY ¼ TO 1¼% IN INCREMENTS OF 1/20%.

THICKNESS OF SLABS IN INCHES AND FEET

11" [.917']		12" [1.00']		15" [1.25']		16" [1.33']		18" [1.50']	
Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area
6100	.330	7200	.360	12000	.45	13000	.480	17000	.540
6600	.396	7800	.432	12900	.54	14000	.576	18300	.648
7100	.462	8400	.504	13800	.63	15000	.672	19600	.756
7600	.528	9000	.576	14700	.72	16000	.768	20900	.864
8100	.594	9600	.648	15600	.81	17000	.864	22200	.972
8600	.660	10200	.720	16500	.90	18000	.960	23500	1.080
9100	.726	10800	.792	17400	.99	19000	1.056	24800	1.188
9600	.792	11400	.864	18300	1.08	20000	1.152	26100	1.296
10100	.858	12000	.956	19200	1.17	21000	1.248	27400	1.404
10600	.924	12600	1.008	20100	1.26	22000	1.344	28700	1.512
11100	.990	13200	1.080	21000	1.35	23000	1.440	30000	1.620
11600	1.056	13800	1.152	21900	1.44	24000	1.536	31300	1.728
12100	1.122	14400	1.224	22800	1.53	25000	1.632	32600	1.836
12600	1.188	15000	1.296	23700	1.62	26000	1.728	33900	1.944
13100	1.254	15600	1.368	24600	1.71	27000	1.824	35200	2.052
13600	1.320	16200	1.440	25500	1.80	28000	1.920	36500	2.160
14100	1.386	16800	1.512	26400	1.89	29000	2.016	37800	2.268
14600	1.452	17400	1.584	27300	1.98	30000	2.112	39100	2.376
15100	1.518	18000	1.656	28200	2.07	31000	2.208	40400	2.484
15600	1.584	18600	1.728	29100	2.16	32000	2.304	41700	2.592
16100	1.650	19200	1.800	30000	2.25	33000	2.400	43000	2.700

continuous simple slabs by multiplying the area of steel by 5.3 and adding the weight of longitudinal reinforcement; the atter should not be less than ⅛%. The proper steel rods, spacing and weights will be found on pages 12 to 15.

SAFE BENDING MOMENTS IN FOOT-POUNDS ON SLABS PER LINEAL FOOT IF REINFORCED BY ¼ TO 1.0% OF STEEL.

THICKNESS OF SLABS IN INCHES AND FEET

20" [1.67']		22" [1.833']		24" [2.00']		27" [2.25']		30" [2.50']	
Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area	Bending Moment Foot Lbs.	Sectional Area
20000	.60	25000	.660	29000	.72	36500	.810	45000	.90
21600	.72	27000	.792	31300	.864	39400	.972	48500	1.08
23200	.84	29000	.924	33600	1.008	42300	1.134	52000	1.26
24800	.96	31000	1.056	35900	1.152	45200	1.296	55500	1.44
26400	1.08	33000	1.188	38200	1.296	48100	1.458	59000	1.62
28000	1.20	35000	1.320	40500	1.440	51000	1.620	62500	1.80
29600	1.32	37000	1.452	42800	1.584	53900	1.782	66000	1.98
31200	1.44	39000	1.584	45100	1.728	56800	1.944	69500	2.16
32800	1.56	41000	1.716	47400	1.872	59700	2.106	73000	2.34
34400	1.68	43000	1.848	49600	2.016	62600	2.268	76500	2.52
35000	1.80	45000	1.980	51900	2.160	65500	2.430	80000	2.70
35600	1.92	47000	2.112	54200	2.304	68400	2.592	83500	2.88
36200	2.04	49000	2.244	56500	2.448	71300	2.754	87000	3.06
36800	2.20	51000	2.376	58800	2.592	74200	2.916	90500	3.24
37400	2.30	53000	2.508	61100	2.736	77100	3.078	94000	3.42
38000	2.40	55000	2.640	63400	2.880	80000	3.240	97500	3.60

For any other thickness of slab or other percentage of reinforcement, use the formula: Moment in foot-pounds per lineal foot of slab=coefficient×t^2, when t thickness of slab

SAFE BENDING MOMENTS IN 1000 FOOT-POUNDS ON SLABS PER LINEAL FOOT IF REINFORCED BY THE MINIMUM PERCENTAGE OF ¼% AND SOME HIGHER PERCENTAGES.

Thickness of Slab in Inches and Feet	Moment in 1000 Foot Lbs.	Area of Steel per Lineal Foot	Thickness of Slab in Inches and Feet	Moment in 1000 Foot Lbs.	Area of Steel per Lineal Foot	Thickness of Slab in Inches and Feet	Moment in 1000 Foot Lbs.	Area of Steel per Lineal Foot
36″ (3′)	65	1.08	60″ (5′)	180	1.80	144″ (12′)	1030	4.32
	75	1.51		252	3.60		1450	8.64
	91	2.16		395	7.20	168″ (14′)	1410	5.04
	117	3.23	66″ (5.5′)	217	1.98		1970	10.08
	143	4.32		305	3.96	192″ (16′)	1840	5.76
40″ (3.33)	80	1.20		478	7.92		2570	11.52
	93	1.68	72″ (6′)	259	2.16	216″ (18′)	2330	6.48
	112	2.40		363	4.32		3260	12.96
	144	3.60	78″ (6.5′)	304	2.34	240″ (20′)	2870	7.20
	176	4.80		425	4.68		4030	14.40
42″ (3.5′)	88	1.26	84″ (7′)	353	2.52	288″ (24′)	4140	8.64
	102	1.76		493	5.04		5800	17.28
	123	2.52	90″ (7.5′)	405	2.70	336″ (28′)	5640	10.08
45″ (3.75′)	101	1.35		567	5.40		7900	20.16
	117	1.89	96″ (8′)	461	2.88	384″ (32′)	7380	11.52
	142	2.70		645	5.76		10300	23.04
48″ (4′)	115	1.44	102″ (8.5′)	520	3.06	432″ (36′)	9350	12.96
	134	2.02		729	6.12		13100	25.92
	161	2.88	108″ (9′)	582	3.24	480″ (40′)	11500	14.4
	253	5.76		817	6.48		16100	28.8
54″ (4.5)	146	1.62	120″ (10′)	720	3.60	540″ (45′)	14600	16.20
	169	2.27		1010	7.20		20400	32.4
	204	3.24	132″ (11′)	870	3.96	600″ (50′)	18000	18.0
	320	6.48		1220	7.92		25200	36.0

n inches. For values of coefficients see explanation of Slab Tables. For ¼%, ½% and ¾% coefficient=50, 70 and 90, respectively.

BENDING MOMENTS IN FOOT-POUNDS IN SIMPLE
LOADED WITH A TOTAL LOAD FROM

Total Load per Square Foot	SPAN IN FEET								
	4	5	5′ 6″	6	6′ 6″	7	7′ 6″	8	8′ 6″
30	48	75	91	108	127	147	169	192	217
100	160	250	303	360	423	490	560	640	723
110	176	275	333	396	465	539	620	704	795
120	192	300	363	432	508	588	676	768	868
130	208	325	394	468	550	637	732	832	940
140	224	350	424	504	592	686	788	896	1012
150	240	375	454	540	634	735	845	960	1084
160	256	400	484	576	671	784	900	1024	1157
170	272	425	514	612	719	833	958	1088	1230
180	288	450	545	648	761	882	1014	1152	1300
190	304	475	575	684	803	931	1070	1216	1373
200	320	500	605	720	845	980	1125	1280	1445
225	360	563	681	810	952	1100	1270	1440	1630
250	400	625	757	900	1058	1230	1410	1600	1810
275	440	683	832	990	1164	1350	1550	1760	1990
300	480	750	908	1080	1268	1470	1690	1920	2168
350	560	875	1060	1260	1480	1720	1970	2240	2530
400	640	1000	1210	1440	1690	1960	2250	2560	2890
500	800	1250	1512	1800	2115	2450	2820	3200	3620
600	960	1500	1815	2160	2540	2940	3370	3840	4340
700	1120	1750	2118	2520	2960	3430	3940	4480	5050
800	1280	2000	2420	2880	3380	3920	4500	5120	5780
900	1440	2250	2722	3240	3800	4410	5060	5760	6500
1000	1600	2500	3025	3600	4225	4900	5625	6400	7225

The bending moments are figured by the formula $\frac{pl^2}{10}$ when **p** total load per square foot, **l** clear span in feet. For slabs freely supported, the bending moments increase by 25%. This table can also be used for finding the bending

SLABS PER LINEAL FOOT OF SLAB, IF UNIFORMLY 30 TO 1000 LBS. PER SQUARE FOOT.

Total Load Per Square Foot or per Lineal Foot	SPAN IN FEET								
	9	9′ 6″	10	10′ 6″	11	11′ 6″	12	12′ 6″	13
30	243	271	300	332	363	397	432	468	508
100	810	903	1000	1103	1210	1320	1440	1560	1690
110	891	993	1100	1213	1331	1460	1584	1720	1860
120	972	1083	1200	1324	1452	1590	1728	1880	2030
130	1053	1174	1300	1434	1573	1720	1872	2030	2200
140	1134	1264	1400	1544	1694	1850	2016	2190	2370
150	1215	1354	1500	1654	1815	1990	2160	2340	2540
160	1296	1444	1600	1764	1936	2120	2304	2500	2710
170	1377	1533	1700	1875	2057	2250	2448	2660	2880
180	1458	1623	1800	1985	2178	2380	2592	2820	3040
190	1539	1714	1900	2095	2299	2520	2736	2970	3220
200	1620	1814	2000	2205	2420	2650	2880	3130	3380
225	1821	2040	2250	2480	2720	2980	3240	3520	3810
250	2025	2260	2500	2760	3030	3320	3600	3910	4220
275	2225	2480	2750	3040	3330	3640	3960	4300	4650
300	2430	2710	3000	3320	3630	3970	4320	4680	5080
350	2830	3160	3500	3860	4240	4630	5020	5480	5920
400	3240	3610	4000	4410	4840	5290	5760	6250	6780
500	4050	4510	5000	5510	6050	6610	7200	7800	8450
600	4860	5410	6000	6610	7260	7930	8640	9380	10140
700	5670	6310	7000	7710	8470	9260	10080	10940	11800
800	6480	7210	8000	8820	9680	10590	11520	12500	13520
900	7290	8110	9000	9920	10890	11900	12960	14060	15210
1000	8100	9025	10000	11025	12100	13225	14400	15625	16900

moments in beams and girders in substituting for the load per square foot, the total load per lineal foot of the beam or girder.

BENDING MOMENTS IN FOOT-POUNDS IN SIMPLE LOADED WITH A TOTAL LOAD OF

Total Load Per Square Foot or per Lineal Foot	SPAN IN FEET								
	13½	14	14½	15	15½	16	16½	17	17½
30	548	588	630	675	720	770	818	867	920
100	1830	1960	2100	2250	2400	2560	2720	2890	3070
110	2010	2160	2310	2470	2650	2820	3000	3180	3380
120	2190	2350	2520	2700	2880	3070	3270	3470	3680
130	2370	2550	2730	2930	3120	3330	3550	3750	3990
140	2550	2740	2940	3150	3360	3580	3820	4050	4300
150	2730	2940	3150	3380	3600	3810	4090	4340	4600
160	2920	3140	3360	3600	3840	4100	4350	4620	4900
170	3100	3330	3570	3830	4100	4350	4630	4910	5200
180	3280	3530	3780	4050	4340	4600	4900	5200	5500
190	3460	3730	3910	4280	4560	4860	5180	5490	5810
200	3650	3920	4200	4500	4800	5100	5440	5780	6110
225	4100	4410	4725	5060	5400	5750	6120	6500	6900
250	4560	4900	5250	5630	6000	6380	6800	7200	7660
275	5010	5400	5775	6200	6600	7030	7500	7920	8400
300	5480	5890	6300	6750	7200	7680	8180	8660	9200
350	6390	6850	7350	7900	8400	8950	9520	10100	10700
400	7300	7830	8400	9000	9600	10200	10900	11600	12300
500	9110	9800	10500	11250	12000	12800	13600	14450	15300
600	10920	11800	12600	13500	14400	15300	16300	17300	18400
700	12780	13700	14700	15800	16800	17900	19100	20200	21500
800	14600	15700	16800	18000	19200	20500	21800	23100	24500
900	16400	17600	18900	20250	21600	23000	24500	26000	27500
1000	18225	19600	21000	22500	24000	25600	27200	28900	30500

The bending moments are figured by the formula $\frac{pl^2}{10}$ To find the proper thickness of slab and the reinforcement required, see pages 16 to 21. The cheapest slab is that, which

SLABS PER LINEAL FOOT OF SLAB, IF UNIFORMLY 30 TO 1000 LBS. PER SQUARE FOOT.

Total Load Per Square Foot or per Lineal Foot	SPAN IN FEET								
	18	18½	19	20	21	22	23	24	25
30	972	1030	1080	1200	1330	1450	1590	1730	1870
100	3240	3430	3610	4000	4410	4840	5290	5760	6250
110	3560	3780	3970	4400	4850	5320	5820	6340	6880
120	3900	4110	4350	4800	5300	5800	6350	6910	7500
130	4210	4450	4700	5200	5740	6280	6890	7500	8110
140	4540	4800	5060	5600	6190	6750	7400	8070	8750
150	4860	5150	5410	6000	6620	7250	7930	8640	9350
160	5190	5480	5780	6400	7060	7720	8460	9210	10000
170	5500	5810	6150	6800	7500	8220	9000	9800	10600
180	5820	6180	6500	7200	7940	8700	9520	10400	11200
190	6150	6500	6860	7600	8390	9200	10080	10950	11800
200	6480	6850	7220	8000	8820	9680	10600	11500	12500
225	7290	7700	8120	9000	9930	10900	11900	13000	14000
250	8100	8550	9120	10000	11000	12100	13250	14400	15600
275	8900	9400	9900	11000	12150	13300	14550	15900	17200
300	9720	10300	10800	12000	13250	14500	15900	17300	18700
350	11300	12000	12600	14000	15450	17000	18500	20150	21800
400	13000	13700	14400	16000	17650	19400	21200	23100	25000
500	16200	17100	18100	20000	22050	24200	26500	28800	31300
600	19440	20500	21600	24000	26450	29000	31800	34600	37500
700	22700	24000	25300	28000	30850	33800	37000	40300	43600
800	25900	27400	28800	32000	35300	38800	42300	46100	49900
900	29200	30800	32400	36000	39700	43500	47600	51900	56100
1000	32400	34300	36100	40000	44100	48400	52900	57600	62500

is reinforced by ¼ to ½%; only, if the cost of steel is very low in comparison with that of concrete, a slab reinforced by ¾% may prove the cheapest slab.

BENDING MOMENTS IN FEET-POUNDS IN SQUARE MAIN DIRECTIONS, IF UNIFORMLY LOADED

Total Load Per Square Foot	SPAN IN FEET										
	4	5	5½	6	6½	7	7½	8	8½	9	9½
30	20	31	38	45	53	61	71	80	91	101	113
100	67	104	126	150	176	205	235	267	302	338	376
110	73	115	138	165	194	225	258	294	332	372	415
120	80	125	151	180	211	245	282	320	362	405	452
130	87	135	164	195	229	265	305	347	393	440	490
140	93	145	177	210	246	286	328	374	422	473	528
150	100	156	189	225	263	306	352	400	452	501	564
160	106	166	202	240	282	326	375	427	482	540	601
170	113	176	215	255	299	347	398	454	513	573	640
180	120	187	227	270	316	367	422	480	543	608	678
190	127	197	240	285	335	388	445	507	573	642	715
200	133	208	252	300	352	409	468	533	603	675	754
225	150	235	283	338	396	460	527	600	680	760	845
250	167	261	315	375	440	510	585	667	754	845	940
275	183	287	346	413	485	562	644	735	830	930	1040
300	200	313	378	450	528	612	702	800	905	1010	1130
350	233	365	440	525	615	714	820	935	1060	1180	1320
400	265	416	505	600	702	816	935	1070	1210	1350	1510
500	333	521	630	750	880	1020	1170	1340	1510	1690	1880
600	400	625	755	900	1060	1230	1410	1600	1810	2020	2260
700	465	730	880	1050	1230	1430	1640	1870	2110	2370	2630
800	530	835	1010	1200	1410	1630	1870	2130	2420	2700	3020
900	600	935	1130	1350	1580	1840	2110	2400	2720	3050	3380
1000	667	1040	1260	1500	1760	2050	2350	2670	3020	3380	3760

The bending moments are figured by the formula $\frac{pl^2}{24}$ when p the total load per square foot and l clear span in feet. Where the slabs are not exactly square, but do not vary in

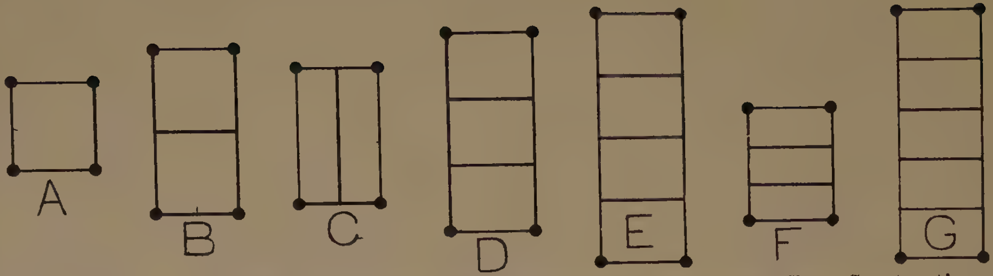
SLABS, PER LINEAL FOOT OF SLAB IN BOTH
WITH 30 TO 1000 POUNDS PER SQUARE FOOT.

Total Load Per Square Foot	SPAN IN FEET										
	10	10½	11	11½	12	12½	13	13½	14	14½	15
30	125	138	151	166	180	195	212	228	245	263	281
100	416	461	505	552	600	652	704	760	815	875	937
110	460	508	555	602	660	718	775	835	900	965	1030
120	500	552	606	662	720	782	845	915	980	1050	1130
130	540	599	656	719	780	848	915	990	1060	1140	1220
140	582	645	707	773	840	915	988	1070	1140	1230	1310
150	623	690	758	830	900	978	1060	1140	1220	1320	1410
160	668	735	809	882	960	1040	1130	1220	1310	1410	1500
170	709	782	860	940	1020	1110	1200	1290	1390	1490	1590
180	750	830	910	992	1080	1170	1270	1370	1470	1580	1690
190	790	875	961	1050	1140	1240	1340	1440	1550	1670	1780
200	834	920	1010	1110	1200	1300	1410	1520	1630	1750	1870
225	935	1030	1140	1240	1350	1470	1590	1710	1840	1970	2110
250	1040	1150	1270	1380	1500	1630	1760	1900	2050	2190	2350
275	1150	1270	1390	1520	1650	1790	1940	2090	2250	2420	2580
300	1250	1380	1520	1660	1800	1950	2120	2280	2450	2640	2820
350	1460	1610	1770	1930	2100	2280	2470	2660	2850	3070	3280
400	1670	1840	2020	2210	2400	2610	2820	3050	3260	3520	3750
500	2080	2300	2530	2760	3000	3250	3520	3800	4080	4390	4700
600	2500	2760	3040	3320	3600	3910	4230	4560	4900	5260	5620
700	2920	3220	3540	3870	4200	4570	4940	5320	5700	6130	6550
800	3330	3680	4050	4420	4800	5210	5620	6080	6520	7010	7500
900	3750	4150	4550	4970	5400	5880	6350	6840	7340	7900	8450
1000	4150	4610	5050	5520	6000	6510	7040	7600	8150	8750	9370

length to width more than the ratio 1:1.333, we substitute for l the mean of the two sides. For proper slabs and reinforcing, see pages 12 to 19. In freely supported slabs the moments increase 25%.

BENDING MOMENTS IN FOOT-POUNDS IN SQUARE SLABS PER LINEAL FOOT OF SLAB IN EACH DIRECTION, IF UNIFORMLY LOADED WITH 30 TO 1000 LBS. PER SQUARE FOOT.

Total Load Per Square Foot	SPAN IN FEET								
	15½	**16**	**16½**	**17**	**18**	**19**	**20**	**22**	**25**
30	300	320	340	361	405	453	500	605	783
100	1000	1070	1130	1200	1350	1510	1670	2020	2610
110	1100	1170	1250	1320	1490	1660	1830	2220	2870
120	1200	1280	1360	1440	1620	1810	2000	2420	3140
130	1300	1390	1470	1560	1760	1960	2170	2630	3390
140	1400	1500	1590	1680	1890	2110	2330	2830	3650
150	1500	1610	1700	1800	2020	2260	2500	3030	3900
160	1600	1710	1810	1920	2160	2420	2670	3230	4160
170	1700	1820	1930	2040	2300	2570	2810	3430	4430
180	1800	1920	2040	2160	2430	2720	3000	3630	4700
190	1900	2030	2150	2280	2570	2850	3170	3830	4950
200	2000	2140	2260	2400	2700	3000	3330	4040	5210
225	2250	2400	2550	2700	3040	3400	3750	4540	5870
250	2500	2670	2830	3000	3370	3770	4190	5040	6500
275	2750	2940	3110	3200	3720	4150	4590	5540	7160
300	3000	3210	3400	3600	4050	4520	5000	6050	7800
350	3500	3750	3960	4200	4720	5270	5830	7050	9110
400	4000	4280	4530	4800	5400	6020	6680	8050	10400
500	5000	5340	5680	6000	6750	7530	8330	10100	13000
600	6000	6400	6800	7200	8100	9030	10000	12100	15700
700	7000	7480	7910	8400	9450	10540	11600	14100	18300
800	8000	8550	9060	9600	10800	12050	13300	16100	20800
900	9000	9650	10200	10800	12100	13500	15000	18100	23500
1000	10000	10700	11300	12000	13500	15100	16700	20200	26100



Figs. 11 and 17. Showing Girder and Beam Arrangement for Typical Floor Construction.

THICKNESS AND AREA OF REINFORCING

Load on Ground per Square Foot in Pounds	3000	4000	5000	6000	8000	10000
Projection in Feet	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel
1′–0″	1500 6″ .18	2000 8″ .20	2500 8″ .20	3000 9″ .30	4000 10″ .30	5000 12″ .30
1′–3″	2350 8″ .20	3140 9″ .30	3930 10″ .30	4720 12″ .30	6280 12″ .36	7850 12″ .55
1′–6″	3370 9″ .30	4500 12″ .30	5620 12″ .36	6730 12″ .40	9000 14″ .40	11300 15″ .50
1′–9″	4600 12″ .30	6140 12″ .36	7680 12″ .55	9210 14″ .40	12300 15″ .50	15400 18″ .50
2′–0″	6000 12″ .30	8000 13″ .36	10000 14″ .40	12000 15″ .50	16000 18″ .50	20000 20″ .50
2′–3″	7600 12″ .40	10200 15″ .40	12700 15″ .60	15300 18″ .50	20300 20″ .60	25400 24″ .70
2′–6″	9500 14″ .40	12600 15″ .60	15700 18″ .50	18900 20″ .60	25200 24″ .70	31500 24″ 1.00
2′–9″	11400 15″ .50	15200 18″ .50	19000 20″ .60	22700 22″ .70	30400 24″ .90	38000 30″ .90
3′–0″	13500 18″ .50	18000 18″ .70	22500 20″ .80	27000 24″ .70	36000 27″ .90	45000 30″ 1.00

The bending moment is figured by the formula $\frac{pl^2}{2}$ when **p** the load on the ground per square foot and l the pro-

PER LINEAL FOOT OF WALL FOOTINGS.

Load on Ground per Square Foot in Pounds	3000	4000	5000	6000	8000	10000
Projection in Feet	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel	Bending Moment Thickness Area of Steel
3′–3″	15800 **18″** .55	21100 **20″** .70	26500 **24″** .70	31700 **24″** .90	42400 **30″** .95	52800 **30″** 1.50
3′–6″	18400 **18″** .75	24500 **24″** .70	30600 **24″** .90	36700 **27″** .95	49000 **30″** 1.20	61500 **36″** 1.20
3′–9″	21200 **24″** .60	28200 **24″** .85	35300 **27″** .90	42300 **30″** .95	56400 **36″** 1.00	70400 **36″** 1.30
4′–0″	24000 **22″** .66	32000 **24″** .86	40000 **27″** .97	48000 **30″** 1.00	64000 **36″** 1.00	80000 **36″** 1.73
4′–6″	30300 **24″** .72	40500 **27″** 1.00	50700 **30″** 1.10	60800 **36″** 1.00	81000 **36″** 1.80	101000 **42″** 1.76
5′–0″	37500 **27″** .81	50000 **30″** 1.00	62500 **36″** 1.00	75000 **36″** 1.50	100000 **42″** 1.75	125000 **48″** 1.50
5′–6″	45300 **30″** .90	60300 **36″** 1.00	75500 **36″** 1.50	90700 **40″** 1.44	121000 **48″** 1.50	151000 **48″** 2.50
6′–0″	54000 **32″** .96	72000 **36″** 1.40	90000 **40″** 1.44	108000 **42″** 1.80	144000 **48″** 2.4	180000 **54″** 2.4
7′–0″	73500 **36″** 1.40	98000 **40″** 1.92	123000 **48″** 1.50	147000 **48″** 2.40	196000 **54″** 3.00	246000 **60″** 3.50

ection in feet. To find the weight of steel per sq. ft. multiply area with 3.4.

PROPERTIES OF

Side in Feet	Load on Ground in 1000 Pounds per Square Foot	Total Load on Footing in 1000 Pounds	Least Side of Column Base Inches	Depth in Center in Inches	Depth at Edge Inches	Area of Steel in one Direction	Cubic Feet of Concrete in Footing	Weight of Steel in Footing
4′	3	48	8	12	6	.72	12	21
	4	64	10	13	6	.78	14	22
	5	80	10	15	6	.90	15	25
	6	96	12	16	6	.96	16	27
	8	128	12	18	6	1.08	17	30
	10	160	12	20	8	1.20	20	33
4′-6″	3	61	10	13	6	.88	16	28
	4	81	10	15	6	1.01	18	32
	5	101	12	17	6	1.15	20	36
	6	121	12	18	6	1.22	21	38
	8	162	14	21	8	1.42	26	44
	10	202	14	23	8	1.56	27	49
5′-0″	3	75	10	14	6	1.05	20	37
	4	100	12	17	6	1.28	24	45
	5	125	12	19	6	1.44	26	51
	6	150	14	20	8	1.50	31	53
	8	200	14	24	8	1.80	34	64
	10	250	16	26	8	1.96	37	70
5′-6″	3	90	10	16	6	1.32	27	51
	4	120	12	18	6	1.50	30	58
	5	150	14	20	8	1.66	35	64
	6	180	14	22	8	1.82	37	70
	8	240	16	26	12	2.15	48	83
	10	300	16	28	12	2.32	50	90
6′	3	108	12	17	6	1.54	33	65
	4	144	12	20	6	1.80	37	75
	5	180	14	22	8	2.00	44	85
	6	216	14	24	8	2.20	47	95
	8	288	16	28	12	2.55	60	107
	10	360	18	31	12	2.80	65	117

Where the column bases are considerably larger than given

COLUMN FOOTINGS

Side in Feet	Load on Ground in 1000 Pounds per Square Foot	Total Load on Footing in 1000 Pounds	Least Side of Column Base Inches	Depth in Center in Inches	Depth at Edge Inches	Area of Steel in one Direction	Cubic Feet of Concrete in Footing	Weight of Steel in Footing
6′-6″	3	126	12	19	6	1.85	42	84
	4	169	14	21	8	2.05	49	94
	5	211	14	24	8	2.35	55	107
	6	253	16	26	12	2.55	66	116
	8	338	18	30	12	2.95	76	134
	10	422	20	34	12	3.35	82	152
7′-0″	3	147	12	20	8	2.10	56	102
	4	196	14	23	8	2.45	61	119
	5	245	16	26	8	2.75	70	133
	6	295	18	28	12	2.95	81	142
	8	392	20	33	12	3.50	92	170
	10	490	22	36	15	3.80	105	184
7′-6″	3	168	14	22	8	2.50	66	130
	4	225	16	25	8	2.85	74	148
	5	282	18	28	8	3.15	82	163
	6	338	20	30	12	3.40	90	175
	8	450	22	35	12	3.95	109	205
	10	560	24	39	15	4.40	132	228
8′	3	192	14	23	8	2.75	78	152
	4	256	16	26	8	3.15	85	174
	5	320	18	30	8	3.60	90	200
	6	384	20	32	12	3.85	114	215
	8	512	24	37	15	4.45	137	245
	10	640	24	42	15	5.05	152	280
8′-6″	3	216	16	24	8	3.05	90	180
	4	288	18	28	8	3.56	100	208
	5	360	20	31	8	3.95	110	230
	6	432	22	34	12	4.35	132	255
	8	578	24	40	15	5.10	156	300
	10	720	26	44	15	5.60	166	330

above, the depth of the footings may be diminished.

PROPERTIES OF

Side in Feet	Load on Ground in 1000 Pounds per Square Foot	Total Load on Footing in 1000 Pounds	Least Side of Column Base Inches	Depth in Center in Inches	Depth at Edge Inches	Area of Steel in one Direction	Cubic Feet of Concrete in Footing	Weight of Steel in Footing
9′	3	243	16	26	8	3.50	106	218
	4	324	18	30	8	4.10	120	255
	5	405	20	33	8	4.50	131	280
	6	486	22	36	12	4.90	153	305
	8	648	25	42	15	5.70	187	355
	10	810	27	47	15	6.30	204	390
9′–6″	3	271	18	27	8	3.90	123	255
	4	362	20	31	8	4.50	135	295
	5	453	22	35	8	5.00	150	325
	6	542	24	39	12	5.60	183	365
	8	723	26	44	15	6.30	216	410
	10	950	30	51	15	7.30	236	475
10′	3	300	18	29	8	4.40	139	300
	4	400	20	33	8	5.00	156	340
	5	500	22	37	8	5.60	173	385
	6	600	24	40	12	6.00	206	410
	8	800	27	47	15	7.10	256	485
	10	1000	30	52	18	7.80	284	530
10′–6″	3	330	18	30	8	4.80	157	345
	4	440	20	35	8	5.60	178	405
	5	550	24	39	8	6.20	200	445
	6	660	26	42	12	6.70	233	480
	8	880	28	49	15	7.80	282	560
	10	1100	32	55	18	8.70	329	625
11′	3	363	20	32	8	5.30	182	400
	4	484	22	36	8	6.00	202	450
	5	605	24	41	8	6.80	226	510
	6	726	27	44	12	7.30	266	550
	8	968	30	51	15	8.50	319	640
	10	1210	32	57	18	9.50	362	715

Where the column bases are considerably larger than given

COLUMN FOOTINGS

Side in Feet	Load on Ground in 1000 Pounds per Square Foot	Total Load on Footing in 1000 Pounds	Least Side of Column Base Inches	Depth in Center in Inches	Depth at Edge Inches	Area of Steel in one Direction	Cubic Feet of Concrete in Footing	Weight of Steel in Footing
12′	3	430	20	34	8	6.10	224	500
	4	575	24	39	8	7.10	256	585
	5	720	27	44	8	7.90	284	650
	6	860	28	48	12	8.70	336	715
	8	1150	32	56	18	10.10	424	830
	10	1440	34	62	18	11.20	464	920
13′	3	510	22	36	8	7.10	282	630
	4	675	26	42	8	8.20	315	730
	5	840	28	48	8	9.40	357	840
	6	1020	30	52	12	10.20	413	905
	8	1350	34	60	18	11.70	521	1040
	10	1690	36	67	21	13.10	592	1160
14′	3	590	24	40	8	8.40	343	810
	4	780	27	46	8	9.70	387	930
	5	980	30	51	8	10.70	433	1030
	6	1180	32	56	12	11.80	506	1130
	8	1550	36	64	18	13.40	631	1290
	10	1960	40	72	21	15.10	725	1450
15′	3	675	26	42	8	9.50	406	970
	4	900	28	49	8	11.10	469	1130
	5	1130	32	55	8	12.40	525	1270
	6	1350	34	60	12	13.50	612	1360
	8	1800	40	70	18	15.80	774	1610
	10	2250	42	78	21	17.60	881	1800
20′	3	1200	32	57	8	17.10	925	2340
	4	1600	36	66	8	19.80	1057	2700
	5	2000	40	73	8	21.90	1167	3000
	6	2400	48	80	12	24.00	1380	3280
	8	3200	56	97	18	29.20	1750	4000
	10	4000	60	106	21	31.80	2020	4350

bove, the depth of the footings may be diminished.

PROPERTIES OF REINFORCED

FOUR ROUND BARS					FOUR SQUARE BARS		DIMEN	
Diameter in Inches	Area and Weight of Four Rods	Gas Pipes		Load on 4 Round Rods at 6750 per Sq." in 1000 Lbs.	Area and Weight of 4 Square Rods	Excess Load for Square Rods over Loads given in Table in 1000 Lbs.	7⅝x8″ *61 ‡27000	9⅝x10″ *96 ‡43000
		Inside Diameter and Length	Total Weight of 4 Pipes					
1/2	.78 2.7	Overlap Rods 30 Times Diameter	4.0	5.3	1.00 3.40	1.5	32	48
5/8	1.22 4.2		8.0	8.2	1.56 5.30	2.3	35	51
3/4	1.76 6.10		13.0	11.8	2.25 7.65	3.3	38	55
7/8	2.40 8.20		18.0	16.2	3.06 10.40	4.5	43	59
1	3.14 10.7	1½″ 2′ 6″	27.	21.0	4.00 13.60	5.9	48	64
1 1/8	3.97 13.5	1½″ 2′ 6″	27.	27.0	5.04 17.2	7.3		70
1 1/4	4.90 16.70	2″ 2′ 9″	40.	33.	6.25 21.3	9.2		76
1 3/8	5.93 20.2	2″ 2′ 9″	40.	40.	7.56 25.7	11.0		
1 1/2	7.06 24.1	2″ 2′ 9″	40.	47.5	9.00 30.6	13.1		
1 5/8	8.24 28.2	2″ 2′ 9″	40.	55.5	10.6 36.0	16.0		
1 3/4	9.6 32.7	2½″ 3′ 0″	70.	64.8	12.3 42.0	18.3		
1 7/8	11.06 37.6	2½″ 3′ 0″	70.	75.0	14.10 48.0	20.5		
2	12.56 42.8	2½″ 3′ 0″	70.	85.0	16.0 54.5	23.5		
Cubic Feet of Concrete per Lineal Foot							.42	.67
Diameter of Round Column of same Areas							8.75	11.1
Diameter of Octagonal Column of same Areas							8.6	10.9

*Denotes the area in square inches of the concrete section as well as the weight of column per lineal foot.

CONCRETE COLUMNS.

ONS OF COLUMNS IN INCHES

1½x12″	13¼x13¾″	15¼x15¾″	17¼x17½″	19¼x19½″	21x21½″	23x23¼″
*138	*182	*240	*305	*375	*450	*530
‡62000	‡81000	‡108000	‡137000	‡169000	‡202000	‡238000
67	**86**					
70	**89**	**116**				
74	**93**	**120**	**149**			
78	**97**	**124**	**153**	**185**	**218**	
83	**102**	**129**	**158**	**190**	**223**	**259**
89	**108**	**135**	**164**	**196**	**229**	**265**
95	**114**	**141**	**170**	**202**	**235**	**271**
102	**121**	**148**	**177**	**209**	**242**	**278**
110	**128**	**155**	**185**	**217**	**249**	**286**
	137	**164**	**192**	**225**	**268**	**293**
	146	**173**	**202**	**234**	**267**	**303**
		183	**212**	**244**	**277**	**313**
			222	**254**	**287**	**323**
.96 13.25 13.0	1.27 15.25 15.0	1.67 17.50 17.2	2.12 19.75 19.4	2.61 21.90 21.5	3.14 24.0 23.7	3.70 26.0 25.5

‡Denotes load carried by the concrete alone.
The black figures denote the column loads in 1000 lbs.

PROPERTIES OF REINFORCED

SIX ROUND BARS					SIX SQUARE BARS		DIMEN	
Diameter in Inches	Area and Weight of Six Rods	Gas Pipe Sleeves		Load Carried by 6 Round Rods at 6750 per Sq.″ in 1000 Lbs.	Area and Weight of 6 Square Rods	Excess Load in 1000 Lbs. for Square Rods over Loads given in Table	17¼x17½″ *305 ‡137000	19¼x19½″ *375 ‡169000
		Inside Diameter and Length	Total Weight of 6 Pipes					
1	4.7 16.1	1½″ 2′ 6″	40.	31.7	6.00 20.4	8.8	169	201
1⅛	5.97 20.3	1½″ 2′ 6″	40.	40.	7.56 25.70	10.7	177	209
1¼	7.36 25.1	2″ 2′ 9″	60.	49.5	9.37 31.80	13.5	186	219
1⅜	8.9 30.3	2″ 2′ 9″	60.	60.	11.34 38.50	16.4	197	229
1½	10.6 36.1	2″ 2′ 9″	60.	71.	13.50 46.00	19.6	208	240
1⅝	12.44 42.3	2″ 2′ 9″	60.	84.	15.80 53.70	22.7	221	253
1¾	14.43 49.1	2½″ 3′ 0″	102.	97.	18.40 62.50	26.9	234	266
1⅞	16.55 56.30	2½″ 3′ 0″	102.	111.	21.10 71.50	30.6	248	280
2	18.84 64.2	2½″ 3′ 0″	102.	127.	24.00 81.60	34.7	264	296
2¼	23.85 81.2	3″ 3′ 6″	158.	161.	30.4 103.00	44.		
Cubic Feet of Concrete per Lineal Foot							2.12	2.61
Diameter of Round Column of same Area							19.75	21.9
Diameter of Octagonal Column of same Area							19.40	21.5

*Denotes the area in square inches of the concrete section, as well as the weight of column per lineal foot.

‡Denotes load carried by the concrete alone.

CONCRETE COLUMNS.

IONS OF COLUMNS IN INCHES						
21x21½" *450 ‡202000	23x23¼" *530 ‡238000	25x25¼" *630 ‡284000	27x27¼" *735 ‡330000	29x29" *840 ‡378000	31x31" *960 ‡431000	33x33" *1090 ‡490000
234	**270**	**316**	**362**	**410**	**463**	**522**
242	**278**	**324**	**370**	**418**	**471**	**530**
252	**288**	**334**	**380**	**428**	**481**	**540**
262	**298**	**344**	**390**	**438**	**491**	**550**
273	**309**	**355**	**401**	**449**	**502**	**561**
286	**322**	**368**	**414**	**462**	**515**	**574**
299	**335**	**381**	**427**	**475**	**528**	**587**
313	**349**	**395**	**441**	**489**	**542**	**601**
329	**365**	**411**	**457**	**505**	**558**	**617**
363	**399**	**445**	**491**	**539**	**592**	**651**
3.14 24.0 23.7	3.70 26.0 25.5	4.4 28.4 27.8	5.11 30.6 30.0	5.85 32.7 32.2	6.70 35.0 34.4	7.60 37.3 36.6

The weight of coils per foot=1/20 side of column in inches. The black figures denote the column loads in 1000 pounds when round bars are used.

PROPERTIES OF REINFORCED

EIGHT ROUND BARS					EIGHT SQUARE BARS		DIMEN	
Diameter in Inches	Area and Weight of Eight Rods	Gas Pipe Sleeves		Load Carried by 8 Round Rods at 6750 per Sq." in 1000 Lbs.	Area and Weight of 8 Square Rods	Excess Load in 1000 Lbs. for Square Rods over Loads given in Table	19¼x19½″ *375 ‡169000	21″x21½″ *450 ‡202000
		Inside Diameter and Length	Total Weight of 8 Pipes					
$1\frac{1}{4}$	9.80 33.40	2″ 2′ 9″	80	66.0	12.50 42.5	18.3	235	268
$1\frac{3}{8}$	11.88 40.4	2″ 2′ 9″	80	80.0	15.10 51.3	21.6	249	282
$1\frac{1}{2}$	14.12 48.0	2″ 2′ 9″	80	95.0	18.0 61.1	26.3	264	297
$1\frac{5}{8}$	16.59 56.4	2″ 2′ 9″	80	112.0	21.10 71.80	30.3	281	314
$1\frac{3}{4}$	19.24 65.4	2½″ 3′ 0″	136	130.0	24.50 83.20	35.5	299	332
$1\frac{7}{8}$	22.09 75.1	2½″ 3′ 0″	136	149.0	28.20 96.0	41.2	318	351
2	25.13 85.4	2½″ 3′ 0″	136	170.0	32.0 109.0	46.4		372
$2\frac{1}{4}$	31.8 108.1	3″ 3′ 6″	212	215.0	40.5 138.0	58.8		
$2\frac{1}{2}$	39.2 133.5	3″ 3′ 6″	212	265.0	50.0 170.0	73.0		
$2\frac{3}{4}$	47.6 161.5	3½″ 4′ 0″	240	322.0	60.5 206.0	87.8		
3″	56.54 192.3	3½″ 4′ 0″	240	382.0				
Cubic Feet of Concrete per Lineal Foot							2.61	3.14
Diameter of Round Column of same Area							21.9	24.0
Diameter of Octagonal Column of same Area							21.5	23.7

Columns	9⅝x10″	11½x12″	13¼x13¾″	15¼x15¾″	17¼x17½″
Require form lumber B.M. per Lin. Ft.	8.1	8.9	9.7	10.7	11.6

CONCRETE COLUMNS.

SIONS OF COLUMNS IN INCHES

23x23¼"	25x25¼"	27x27¼"	29x29"	31x31"	33x33"	35x35"
*530	*630	*735	*840	*960	*1090	*1225
‡238000	‡284000	‡330000	‡378000	‡431000	‡490000	‡550000
304	350	396	444	497	556	616
318	364	410	458	511	570	630
333	379	425	473	526	585	645
350	396	442	490	543	602	662
368	414	460	508	561	620	680
387	433	479	527	580	639	699
408	454	500	548	601	660	720
453	496	545	593	646	705	765
	546	595	643	696	755	815
			700	753	812	872
					872	932
3.70 26.0 25.5	4.4 28.4 27.8	5.11 30.6 30.0	5.85 32.7 32.2	6.70 35.0 34.4	7.60 37.3 36.6	8.5 39.4 38.7

9½x19½"	21x21½"	23x23¼"	25x25¼"	27x27¼"	29x29"	31x31"	33x33"
14.0	15.0	18.0	19.0	20.4	21.8	23.	24.3

PROPERTIES OF

DIAMETER						12"	13.5"	16"
Area of round Section, square inches						113	143	201
Area of octagonal Section, square inches						119	150	212
Outside diameter of Helix inches						10	11.5	14
Area of core of Helix						78	104	154
Load carried by core at 720 lbs. per sq. inch						56000	75000	111000
Thickness of Helical Reinforcement	Area Weight	Pitch of helical Reinforcement in fraction of Diameter of Helix	Weight of Helix per foot	Ideal longitudinal Area	Load due to Helix 1000 lbs.	Load on		
1"/4	.0491	1/8	4.2	1.24	32.2	107	133	177
	.167	1/10	5.25	1.55	40.2	115	141	185
		1/12	6.3	1.87	48.5	123	149	193
5/16	.0767	1/8	6.5	1.92	49.8	124	151	195
	.261	1/10	8.15	2.4	62.2	137	163	207
		1/12	9.75	2.88	75.0		176	220
3/8	.1104	1/8	9.4	2.76	71.5		172	216
	.375	1/10	11.8	3.45	89.5			234
		1/12	14.1	4.15	107.5			252
7/16	.1503	1/8	12.9	3.8	98.6			243
	.511	1/10	16.2	4.75	123.4			268
		1/12	19.4	5.7	148.0			
1/2	.1963	1/8	16.7	4.9	127.0			
	.667	1/10	21.0	6.15	160.0			
		1/12	25.	7.35	191.0			
5/8	.306	1/5	26.0	7.7	200.0			
	1.043	1/10	32.5	9.65	250.0			
		1/12	39.0	11.5	298.0			
3/4	.4418	1/8	37.7	11.0	285.0			
	1.502	1/10	47.2	13.7	355.0			
		1/12	56.8	16.5	429.0			
Longitudinal Reinforcement						4-3/4	4-7/8	4-1"
Area of " "						1.76	2.40	3.14
Weight per lin. foot of Longitudinal Reinforcement						6.10	8.2	10.7
Load carried by Longitudinal Reinforcem't at 10800 lbs. per sq. inch						19000	26000	34000

Use 1:3 concrete mixture for these columns.

CONSIDERE COLUMNS.

18.5″	20″	23.3″	26″	28″	32.5″
269	314	426	531	615	830
283	331	446	560	649	871
16.5	18	21.3	24	26	30.5
214	255	355	452	531	730
154000	183000	255000	325000	382000	525000

Columns in 1000 pounds

220	266	340	421	493	636
228	274	348	429	501	644
236	282	356	437	509	652
238	283	357	438	510	653
250	296	370	451	523	666
263	309	383	464	536	679
259	305	379	460	532	675
278	323	397	478	550	693
295	341	415	496	568	711
286	332	406	487	559	702
311	357	431	512	584	727
336	382	456	537	609	752
315	361	435	516	588	731
348	394	468	549	621	764
	425	499	580	652	795
	434	508	589	661	804
	486	550	639	711	854
		607	687	759	902
			674	746	889
			744	816	959
			818	890	1033

4–1″	4–1¼	4–1¼	6–1⅛	6–1¼	6–1¼
3.14	4.90	4.90	5.97	7.36	7.36
10.7	16.70	16.7	20.3	25.1	25.1
34000	53000	53000	64000	79000	79000

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	100	150	175	200	225	250	300	350	400	500
Column Centers	12′x12′ TYPE A,—Square Panels—4 Supports 4″x4″—12′ Ceiling										12′x14′ TYPE A,—Square Slabs—4 Supports 4″x4″—12′ Ceiling									
Number of Girder	5	7	8	8	8	9	10	15	15	17	7	12	12	13	22	22	23	24	25	26
Number of Beam	5	7	8	8	8	9	10	15	15	17	6	11	12	12	13	13	14	16	17	18
Thickness of Slab	3½″	4	4½	4½	5	5	5½	6	6½	6½	4	4½	5	5	5	5½	6	6	6½	7
Weight of Steel in Slab	.91	1.25	1.14	1.67	1.26	1.52	1.67	1.85	1.64	2.60	1.01	1.14	.126	1.51	1.80	1.67	1.80	2.40	2.30	2.95
Average per Sq. Ft. Concrete C. Ft.	.349	.390	.432	.432	.474	.494	.535	.594	.636	.636	.384	.440	.482	.482	.488	.529	.586	.596	.638	.693
Steel Lbs.	1.53	2.15	2.46	2.99	2.58	3.12	3.47	3.65	3.44	4.95	1.75	2.09	2.30	2.65	3.17	3.04	3.38	4.33	4.49	5.44
Form Lumber	2.85	2.85	2.85	2.85	3.01	3.08	3.08	3.08	3.08	3.08	2.89	3.00	3.00	3.00	3.08	3.08	3.13	3.15	3.15	3.23
Column Centers	12′x16′ TYPE A,—Square Slabs—4 Supports 4″x4″—12′ Ceiling										12′x18′ TYPE B,—Square Slabs—4 Supports 4″x4″—12′ Ceiling									
Number of Girder	11	13	14	15	16	17	18	26	27	35	22	24	26	27	34	35	43	49	51	54
Number of Beam	7	11	12	13	14	14	23	23	24	26	4	7	7	7	11	12	12	13	14	16
Thickness of Slab	4″	4½	5	5	5½	5½	6	6½	7	7	3″	3½	4	4	4½	4½	5	5	5½	6
Weight of Steel in Slab	1.01	1.30	1.51	1.94	1.68	2.18	2.40	2.50	2.60	3.87	.85	1.10	1.01	1.26	1.16	1.36	1.30	1.80	1.70	2.10
Average per Sq. Ft. Concrete C. Ft.	.390	.437	.479	.491	.532	.532	.589	.654	.695	.716	.329	.386	.441	.451	.502	.502	.554	.564	.626	.684
Steel Lbs.	1.80	2.30	2.73	3.34	3.32	3.96	4.42	4.52	4.87	6.54	1.95	2.74	2.95	3.35	3.26	3.78	3.72	4.44	4.76	5.91
Form Lumber	2.91	2.95	2.95	2.95	2.95	2.95	3.08	3.17	3.22	3.32	3.28	3.32	3.35	3.35	3.35	3.35	3.41	3.46	3.49	3.53

The numbers of beams and girders refer to Beam Tables, pages 6 to 8.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	100	150	175	200	225	250	300	350	400	500
Column Centers	12'x20' TYPE B,—Square Slabs—7 Supports 4"x4"—12' Ceiling										12'x24' TYPE B,—Square Slabs—7 Supports 4"x4"—12' Ceiling									
Number of Girder	16	26	33	35	36	37	45	53	54	63	26	36	37	37	45	53	60	63	65	70
Number of Beam	5	7	11	11	12	12	20	21	29	30	6	7	11	12	12	13	14	15	23	31
Thickness of Slab	3½	4	4	4½	4½	5	5	5½	6	6½	3½	4	4½	4½	5	5	5½	6	6	6½
Weight of Steel in Slab	.90	1.01	1.44	1.36	1.65	1.26	2.00	1.93	1.82	2.30	.91	1.25	1.14	1.67	1.26	1.52	1.67	1.85	1.64	2.60
Average per Sq. Ft. Concrete C. Ft.	.373	.437	.455	.497	.513	.555	.575	.628	.696	.767	.390	.457	.506	.506	.561	.573	.628	.697	.748	.810
Steel Lbs.	2.31	2.89	3.32	3.57	4.16	4.19	4.93	4.98	5.27	5.88	2.58	3.54	3.85	4.49	4.08	4.74	4.97	5.25	5.54	6.64
Form Lumber	3.05	3.14	3.29	3.29	3.32	3.32	3.47	3.53	3.63	3.74	2.98	3.09	3.15	3.15	3.20	3.26	3.32	3.56	3.62	3.65
Column Centers	14'x14' TYPE A.—Square Slabs—4 Supports 4"x4"—12' Ceiling										14'x16' TYPE A,—Square Slabs—4 Supports 4"x4"—12' Ceiling									
Number of Girder	7	9	10	15	15	16	17	25	26	34	8	10	15	16	24	24	26	33	34	36
Number of Beam	7	9	10	15	15	16	17	25	26	34	11	13	14	15	16	17	18	26	27	35
Thickness of Slab	4	4½	5	5	5½	6	6	6½	7	7	4	4½	5	5½	6	6	6½	7	7	7½
Weight of Steel in Slab	1.01	1.68	1.52	2.05	1.85	1.68	2.44	2.50	2.60	4.00	1.42	1.84	2.05	1.95	1.70	2.15	2.45	2.85	3.55	4.20
Average per Sq. Ft. Concrete C. Ft.	.383	.441	.423	.497	.538	.580	.580	.638	.704	.716	.385	.435	.484	.533	.583	.583	.637	.704	.704	.777
Steel Lbs.	1.79	3.04	3.06	3.59	3.39	3.45	4.45	4.51	4.89	6.55	2.33	3.12	3.43	3.52	3.37	3.92	4.60	5.00	5.95	7.03
Form Lumber	2.75	2.82	2.82	2.90	2.90	2.95	2.95	3.05	3.14	3.29	2.75	2.78	2.82	2.84	2.99	2.99	3.01	3.20	3.20	3.28

The weight of floor construction=average concrete per square foot×144.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500
Column Centers	14'x18' TYPE C,—Simple Slabs—6 Supports 4"x4"—12' Ceiling									
Number of Girder	19	21	22	23	23	30	32	33	34	43
Number of Beam	10	23	31	32	40	41	42	43	49	53
Thickness of Slab	3	3½	3½	4	4	4	4½	5	5	5½
Weight of Steel in Slab	.53	.67	.90	.80	1.06	1.27	1.16	1.11	1.44	1.60
Average per Sq. Ft. Concrete C. Ft.	.346	.400	.504	.482	.489	.501	.571	.613	.620	.692
Steel Lbs.	2.13	2.63	3.17	3.38	3.64	3.95	4.44	4.82	5.47	6.29
Form Lumber	3.24	3.31	3.37	3.39	3.43	3.57	3.62	3.62	3.66	3.77
Column Centers	14'x22' TYPE B.—Square Slabs—7 Supports 4"x4"—12' Ceiling									
Number of Girder	39	42	48	49	56	58	62	63	64	70
Number of Beam	7	8	12	13	14	14	16	16	24	26
Thickness of Slab	3½	4	4½	4½	5	5	5½	6	6½	6½
Weight of Steel in Slab	1.00	1.52	1.44	1.70	1.52	1.93	2.00	2.10	2.03	3.35
Average per Sq. Ft. Concrete C. Ft.	.384	.441	.500	.486	.552	.572	.639	.581	.733	.794
Steel Lbs.	2.45	3.52	3.33	4.09	3.76	4.52	4.93	5.36	5.53	7.39
Form Lumber	3.18	3.20	3.31	3.31	3.37	3.39	3.46	3.51	3.57	3.46

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500
Column Centers	14'x20' TYPE C,—Simple Slabs—6 Supports 4"x4"—12' Ceiling									
Number of Girder	20	22	23	24	25	26	33	42	43	49
Number of Beam	22	24	25	33	33	34	43	48	52	59
Thickness of Slab	3	3½	3½	4	4	4	4½	5	5	5½
Weight of Steel in Slab	.53	.67	.90	.80	1.06	1.27	1.16	1.11	1.44	1.60
Average per Sq. Ft. Concrete C. Ft.	.345	.395	.421	.477	.477	.501	.565	.631	.644	.711
Steel Lbs.	2.19	2.87	3.37	3.62	4.12	4.79	4.72	4.85	5.70	6.30
Form Lumber	3.24	3.24	3.24	3.30	3.30	3.34	3.53	3.62	3.67	3.83
Column Centers	14'x24' TYPE B,—Square Slabs—7 Supports 4"x4"—12' Ceiling									
Number of Girder	33	48	49	51	59	62	63	68	70	77
Number of Beam	7	8	9	9	10	15	23	24	31	33
Thickness of Slab	3½	4	4½	4½	5	5	5½	6	6½	7
Weight of Steel in Slab	1.26	1.85	1.70	2.10	1.85	2.30	2.27	2.35	2.35	3.00
Average per Sq. Ft. Concrete C. Ft.	.389	.448	.500	.518	.572	.593	.643	.705	.780	.851
Steel Lbs.	2.86	3.79	4.02	4.62	4.69	5.14	5.30	5.52	5.97	7.20
Form Lumber	3.05	3.14	3.17	3.19	3.24	3.34	3.40	3.40	3.41	3.49

The average of form lumber includes all supports, waste and braces.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	100	150	175	200	225	250	300	350	400	500
Column Centers	14′x26′ TYPE B,—Square Slabs—9 Supports 4″x4″—14′ Ceiling										14′x28′ TYPE B,—Square Slabs—9 Supports 4″x4″—14′ Ceiling									
Number of Girder	41	52	53	59	62	63	65	70	71	83	48	58	62	66	67	68	70	81	83	84
Number of Beam	7	8	9	20	21	22	23	24	25	27	7	12	13	14	22	22	24	25	26	34
Thickness of Slab	3½″	4½	4½	5	5	5½	6	6	6½	7	4″	4½	4½	5	5½	5½	6	6½	6½	7
Weight of Steel in Slab	1.43	1.50	1.85	1.70	2.10	1.90	2.10	2.70	2.70	3.50	1.10	1.60	2.20	1.85	1.70	2.27	2.35	2.35	3.20	4.00
Average per Sq. Ft. Concrete C. Ft.	.397	.506	.515	.571	.584	.625	.680	.726	.768	.850	.444	.521	.534	.596	.642	.642	.723	.765	.804	.892
Steel Lbs.	3.00	3.69	4.53	4.16	4.65	4.76	5.51	6.24	6.65	7.93	2.77	3.80	4.71	4.25	4.32	5.09	5.83	5.95	7.50	8.47
Form Lumber	3.23	3.30	3.33	3.47	3.52	3.52	3.57	3.45	3.45	3.58	3.21	3.33	3.40	3.26	3.31	3.31	3.31	3.41	3.46	3.70

Total Load per Sq. Ft.	125	150	175	200	250	125	150	175	250	125	150	175	250	125	150	175	250
Column Centers	14′x 30′-B,-Sq.S.-10S.-18′					14′x35′-D-Sq.S.-11S.-20′				14′x40′-D,-Sq.S.-11S.-20′				14′x50′-E,-Sq.S.-16S.-20′			
Number of Girder	62	67	68	75	77	73	75	81	82	80	82	84	84	85	86	89	92
Number of Beam	19	20	21	22	23	11	12	13	13	19	19	20	21	7	8	12	13
Thickness of Slab	4	4½	4½	5	5½	3½	4	4½	4½	3½	4	4½	5	4	4	4½	5
Weight of Steel in Slab	1.21	1.40	1.85	1.70	2.10	1.30	1.22	1.20	1.60	1.44	1.44	1.44	1.35	1.08	1.41	1.41	1.67
Average per Sq. Ft. Concrete C. Ft.	.495	.557	.557	.625	.691	.502	.543	.585	.613	.500	.571	.653	.695	.603	.603	.692	.809
Steel Lbs.	3.51	3.79	4.51	4.70	5.69	3.71	3.97	4.45	5.14	4.21	4.71	5.10	5.10	5.37	6.60	6.50	6.86
Form Lumber	3.68	3.54	3.54	3.54	3.56	4.18	4.18	4.23	4.26	4.11	4.14	4.25	4.25	4.27	4.27	4.42	4.54

The girders in 14′x30′ to 14′x50′ were figured as freely supported.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	100	150	175	200	225	250	300	350	400	500
Column Centers	16′x16′ TYPE A,—Square Slabs—4 Supports 4″x4″—12′ Ceiling										16′x18′ TYPE C,—Simple Slabs—6 Supports 4″x4″—12′ Ceiling									
Number of Girder	12	22	23	24	25	31	40	41	42	49	19	28	29	30	31	31	40	41	42	48
Number of Beam	12	22	23	24	25	31	40	41	42	49	29	39	40	41	42	43	50	51	53	60
Thickness of Slab	4	5	5½	5½	6	6	6½	7	7	7½	3½	4	4	4½	4½	4½	5	5½	5½	6
Weight of Steel in Slab	1.85	1.85	1.70	2.25	2.20	2.70	2.94	3.20	4.20	4.90	.56	.80	1.16	1.00	1.20	1.50	1.45	1.50	1.80	2.15
Average per Sq. Ft. Concrete C. Ft.	.386	.479	.542	.542	.584	.594	.676	.717	.717	.775	.396	.472	.484	.529	.529	.529	.618	.673	.673	.740
Average per Sq. Ft. Steel Lbs.	2.68	3.05	3.05	3.80	3.96	4.25	4.70	5.20	6.42	7.55	1.99	2.73	3.40	3.52	4.02	4.44	4.62	5.17	5.86	6.50
Average per Sq. Ft. Form Lumber	2.70	2.82	2.82	2.82	2.99	3.13	3.18	3.18	3.26	3.26	3.38	3.54	3.56	3.56	3.56	3.56	3.72	3.74	3.74	3.86
Column Centers	16′x20′ TYPE C,—Simple Slabs—6 Supports 4″x4″—12′ Ceiling										16′x22′ TYPE C,—Simple Slabs—6 Supports 4″x4″—12′ Ceiling									
Number of Girder	21	23	30	31	32	33	41	43	48	51	28	38	39	40	41	42	43	49	51	54
Number of Beam	30	32	41	42	43	48	52	53	59	63	30	40	46	47	48	49	51	54	62	64
Thickness of Slab	3½	4	4	4½	4½	4½	5	5½	5½	6	3½	4	4	4½	4½	4½	5	5½	5½	6
Weight of Steel in Slab	.56	.80	1.16	1.00	1.20	1.50	1.45	1.50	1.80	2.15	.56	.80	1.16	1.00	1.20	1.50	1.45	1.50	1.80	2.15
Average per Sq. Ft. Concrete C. Ft.	.392	.462	.478	.523	.548	.554	.624	.665	.689	.771	.420	.486	.497	.564	.564	.564	.617	.674	.721	.763
Average per Sq. Ft. Steel Lbs.	2.31	3.11	3.56	3.72	4.24	4.67	4.85	5.68	5.70	7.00	2.25	3.03	3.59	3.77	4.28	4.96	5.35	5.84	6.36	7.72
Average per Sq. Ft. Form Lumber	3.30	3.31	3.47	3.47	3.51	3.54	3.66	3.66	3.77	3.84	3.40	3.50	3.52	3.58	3.58	3.58	3.59	3.67	3.77	3.77

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	125	150	200	250	300	125	150	200	250	300	125	150	200	250	300	125	150	200	250	300
Column Centers	16'x28'-B,-Sq. S.-11S.-14'					16'x30'-B,-Sq- S.-11S-18'					16'x35'-D,-Sq.S.-14 S. 20'					16'x40'-D,-Sq. S.-14 S.-20'				
Number of Girder	57	59	64	69	77	66	73	76	83	84	79	81	84	85	86	82	84	85	86	95
Number of Beam	20	21	23	24	26	20	22	23	25	32	19	20	22	23	31	13	21	22	24	32
Thickness of Slab	4	4½	4½	5	5½	4	4½	5	5½	6	4	4½	4½	5	5½	4	4½	5	5½	5½
Weight of Steel in Slab	1.26	1.36	1.85	2.10	2.55	2.16	1.64	1.76	2.18	2.50	1.05	1.20	1.65	1.95	2.20	1.25	1.44	1.36	1.70	2.55
Average per Sq. Ft. Concrete C. Ft.	.452	.510	.536	.595	.683	.496	.561	.631	.690	.776	.525	.567	.629	.687	.735	.541	.623	.665	.720	.870
Average per Sq. Ft. Steel Lbs.	3.12	3.63	4.69	5.06	6.21	4.11	3.97	4.73	5.88	6.20	3.38	4.03	5.22	6.20	7.27	4.35	4.79	5.41	6.62	6.96
Average per Sq. Ft. Form Lumber	3.36	3.36	3.40	3.40	3.40	3.45	3.47	3.48	3.54	3.72	3.72	3.72	3.84	3.84	3.92	3.62	3.78	3.78	3.78	4.13
Total Load per Sq. Ft.	150	175	200	250	300															
Column Centers	16'x 50'-E,-Sq.S.-19S.-20'					18'x30'-F-S. S.-19S.-20'					18'x35'-D,-Sq.S.-19S.-20'					18'x40'-D,-Sq.P.-19S.-20'				
Number of Girder	90	91	92	93	94	60	67	70	77	83	76	82	84	86	89	83	84	90	91	92
Number of Beam	12	13	21	29	30	21	23	31	33	41	21	28	30	31	32	14	15	24	26	33
Thickness of Slab	4	4½	4½	5	5½	4	4½	5	5½	6	4½	4½	5½	6	6½	4½	4½	5½	6	6½
Weight of Steel in Slab	1.05	1.20	1.65	1.95	2.20	.90	1.01	1.27	1.58	1.75	1.14	1.26	1.40	1.85	2.00	1.18	1.60	1.69	2.10	2.28
Average per Sq. Ft. Concrete C. Ft.	.681	.723	.729	.832	.873	.472	.558	.628	.700	.765	.563	.597	.711	.753	.844	.564	.605	.787	.841	.892
Average per Sq. Ft. Steel Lbs.	4.34	5.19	6.26	7.69	9.07	3.60	3.87	4.81	5.74	6.16	3.82	4.16	4.78	6.40	6.74	4.35	4.86	5.08	6.28	7.02
Average per Sq. Ft. Form Lumber	4.02	4.02	4.11	4.22	4.22	3.86	3.78	3.88	3.93	4.05	3.90	3.99	4.11	4.14	4.21	3.64	3.74	3.95	3.99	4.05

The girders for 16'x35' to 18'x40' were assumed freely supported.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	125	150	200	250	300					
Column Centers	16′x24′ TYPE C,—Simple Slabs—8 Supports 4″x4″—12′ Ceiling										16′x26′ TYPE C,—Simple Slabs—8 Supports 4″x4″—14′ Ceiling									
Number of Girder	29	32	40	41	42	43	49	51	54	63	32	34	43	52	53					
Number of Beam	38	41	47	48	49	52	58	62	63	65	40	41	48	57	59					
Thickness of Slab	3½	4	4	4½	4½	4½	5	5½	5½	6	3½	3½	4	4½	5					
Weight of Steel in Slab	.56	.80	1.16	1.00	1.20	1.50	1.45	1.50	1.80	2.15	.56	.90	1.10	1.20	1.20					
Average per Sq. Ft. Concrete C. Ft.	.420	.494	.559	.559	.559	.569	.637	.719	.719	.805	.453	.453	.519	.609	.669					
Steel Lbs.	2.39	3.48	3.84	4.03	4.60	5.20	5.40	5.96	7.09	7.73	3.08	3.95	4.54	5.08	5.94					
Form Lumber	3.42	3.47	3.60	3.60	3.60	3.62	3.71	3.78	3.78	3.94	3.48	3.48	3.59	3.75	3.75					
Total Load per Sq. Ft.											100	150	175	200	225	250	300	350	400	500
Column Centers	18′x18′ TYPE A,—Square Slabs—5 Supports 4″x4″—12′ Ceiling										18x′20′ TYPE F,—Simple Slabs—10 Supports 4″x4″—12′ Ceiling									
Number of Girder	15	18	26	27	34	41	43	49	51	54	26	42	48	52	51	53	60	67	68	71
Number of Beam	15	18	26	27	34	41	43	49	51	54	11	20	28	29	29	30	31	32	41	42
Thickness of Slab	5	6	6	6½	6½	7	7	7½	8	8	3	3½	3½	4	4	4	4½	4½	5	5½
Weight of Steel in Slab	1.35	1.60	2.25	2.25	2.70	2.85	3.90	4.40	4.45	6.50	.55	.60	.70	.70	.81	.92	1.04	1.30	1.14	1.30
Average per Sq. Ft. Concrete C. Ft.	.480	.563	.594	.636	.648	.702	.702	.759	.828	.828	.360	.426	.472	.527	.527	.527	.578	.634	.694	.755
Steel Lbs.	2.55	3.38	4.03	4.23	4.12	4.63	6.13	6.76	7.12	9.80	2.62	2.82	3.10	3.61	3.83	4.29	4.84	5.21	5.50	6.53
Form Lumbr	2.84	2.84	2.95	2.95	3.06	3.14	3.14	3.22	3.25	3.25	3.39	3.59	3.78	3.80	3.80	3.80	3.80	3.80	3.90	3.91

TYPICAL FLOOR CONSTRUCTION.

otal Load per Sq. Ft.	100	150	175	200	225	250	300	350	400	500	100	150	175	200	225	250	300	350	400	500
olumn Centers	18′x22′ TYPE F,—Simple Slabs—10 Supports 4″x4″—12′ Ceiling										18′x24′ TYPE F,—Simple Slabs—10 Supports 4″x4″—12′ Ceiling									
umber of Girder	41	48	51	53	54	60	64	69	76	82	42	45	59	62	63	64	69	71	82	84
umber of Beam	19	21	29	29	30	31	32	40	41	43	19	28	29	30	31	38	40	41	42	48
hickness of Slab	3	3½	4	4	4½	4½	5	5	5½	6	3½	4	4	4½	4½	5	5	5½	6	6
eight of Steel in Slab	.60	.85	.70	.95	.80	1.04	.90	1.30	1.30	1.50	.60	.70	1.1	1.0	1.30	1.0	1.50	1.50	1.40	2.15
Average per Sq. Ft. Concrete C. Ft.	.377	.426	.516	.516	.558	.567	.647	.673	.750	.803	.413	.501	.517	.569	.569	.623	.667	.728	.791	.838
Steel Lbs.	2.45	3.14	3.59	4.00	4.21	4.68	4.96	5.36	5.81	6.81	2.49	3.44	4.04	4.13	4.79	4.49	5.40	6.05	6.20	7.18
Form Lumber	3.42	3.46	3.60	3.60	3.60	3.64	3.72	3.72	3.72	3.77	3.31	3.44	3.62	3.66	3.66	3.75	3.75	3.79	3.79	3.95
Total Load per Sq. Ft.											125	150	200	250	300					
Column Centers	18′x26′ TYPE B.—Square Panels—12 Supports 4″x4″—14′ Ceiling										18′x28′ TYPE B,—Square Panels—12 Supports 4″x4″—16′ Ceiling									
Number of Girder	58	59	63	64	69	70	77	83	84	85	53	60	65	71	82					
Number of Beam	14	15	23	30	31	32	33	35	36	37	15	23	24	32	34					
Thickness of Slab	4½	5	5½	6	6	6	6½	7	7½	8	4½	5	5½	6	6½					
Weight of Steel in Slab	1.20	1.60	1.70	1.60	2.10	2.50	2.70	3.00	3.20	4.10	1.25	1.35	1.80	2.25	2.70					
Average per Sq. Ft. Concrete C. Ft.	.498	.551	.610	.658	.674	.710	.763	.815	.902	.944	.456	.555	.606	.704	.768					
Steel Lbs.	3.28	3.94	4.21	4.28	4.90	5.59	6.17	6.99	7.35	9.19	3.53	3.79	4.69	5.48	6.26					
Form Lumber	3.26	3.29	3.38	3.46	3.46	3.46	3.46	3.46	3.65	3.65	3.43	3.52	3.58	3.58	3.61					

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	125	150	200	250	300	125	150	200	300	350	125	150	200	300	350
Column Centers	18′x50′-Type E,-Sq. Slabs-26Spts.-20′C.					20′x20′-Type F,-Sl. Slabs-10 Spts.-16′C.					20′x′22-Type F,-Sl. Slabs-10 Spts.-16′C.				
Number of Girder	86	89	93	94	98	20	28	30	33	41	21	29	31	34	42
Number of Beam	14	22	30	32	33	42	44	51	63	65	43	45	53	65	70
Thickness of Slab	4½	4½	5½	6	6½	3	3	3½	4	4½	3	3½	4	4½	5
Weight of Steel in Slab	1.14	1.26	1.40	1.85	2.00	.53	.57	.69	1.06	1.14	.59	.59	.69	1.20	1.17
Average per Sq. Ft. Concrete C. Ft.	.597	.635	.833	.891	.959	.378	.428	.478	.566	.626	.371	.459	.508	.595	.685
Average per Sq. Ft. Steel Lbs.	5.45	5.57	6.67	8.29	8.59	2.65	3.08	3.73	5.20	5.62	2.89	3.46	3.98	5.58	5.70
Average per Sq. Ft. Form Lumber	3.83	3.98	4.17	4.19	4.24	3.75	3.91	3.95	4.05	4.16	3.60	3.75	3.79	3.89	3.89
Total Load per Sq. Ft.	125	150	200	300	350										
Column Centers	20′x24′-Type F,-Sl. Slabs-10 Spts.-16′C.					20′x26′-Type F-Sl. Slabs-10 Spts.-16′C.					20′x28′-Type F,-Sl. Slabs-13 Spts.-16′C.				
Number of Girder	28	29	38	41	43	28	30	32	42	44	29	30	39	43	48
Number of Beam	52	53	63	70	77	58	60	64	77	83	59	63	70	83	84
Thickness of Slab	3½	3½	4½	5	5½	3½	4	4½	5½	6	4	4½	5	5½	6
Weight of Steel in Slab	.59	.90	.75	1.17	1.23	.79	.70	1.02	1.25	1.33	.69	.75	.85	1.70	1.75
Average per Sq. Ft. Concrete C. Ft.	.459	.459	.572	.673	.724	.460	.501	.575	.714	.791	.492	.546	.631	.714	.798
Average per Sq. Ft. Steel Lbs.	2.90	3.66	3.84	5.27	6.06	3.12	3.69	4.58	5.60	6.43	3.31	3.68	4.28	6.38	6.22
Average per Sq. Ft. Form Lumber	3.68	3.68	3.82	3.76	3.78	3.71	3.71	3.78	3.78	3.85	3.82	3.85	3.85	3.91	4.04

The girders for 18′x50′ were assumed as freely supported.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	125	150	200	300	350	125	150	200	300	350	125	150	200	300	350
Column Centers	20′x30′–Type F,–Sl. Slabs–13 Spts–20′C.					20′x40′–Type E,–Sl. Slabs–18 Spts–20′C.					20′x50′–Type G,–Sl. Slabs–23 Spts–20′C.				
Number of Girder	43	52	58	65	70	23	31	33	44	49	23	31	33		
Number of Beam	51	58	63	71	77	78	80	83	85	86	91	92	93		
Thickness of Slab	4	4½	5	6	6	4	4½	5	6	6	4	4½	5		
Weight of Steel in Slab	.91	1.01	1.27	1.70	2.25	.91	1.01	1.27	1.70	2.25	.91	1.01	1.27		
Average per Sq. Ft. Concrete C. Ft.	.489	.574	.639	.762	.832	.517	.570	.653	.799	.789	.650	.700	.796		
Average per Sq. Ft. Steel Lbs.	2.96	3.19	3.77	5.19	5.74	3.69	4.18	5.37	6.94	7.91	4.92	5.71	7.15		
Average per Sq. Ft. Form Lumber	3.77	3.80	3.98	3.98	3.98	3.73	3.83	3.88	4.04	4.08	3.88	4.08	4.13		
Column Centers	22′x22′–Type F,–Sl. Slabs–10 Spts–16′C.					22′x24′–Type F,–Sl. Slabs–10 Spts–16′ C.					22′x26′–Type F,–Sl. Slabs–13 Spts–16′C.				
Number of Girder	48	52	54	65	70	51	54	64	71	83	59	63	69	83	84
Number of Beam	29	30	39	42	48	23	31	40	43	49	24	25	40	48	52
Thickness of Slab	3	3½	4	4½	5	3½	3½	4½	5	5½	3½	4	4½	5½	6
Weight of Steel in Slab	.59	.59	.69	1.20	1.17	.59	.90	.75	1.17	1.23	.79	.70	1.02	1.25	1.33
Average per Sq. Ft. Concrete C. Ft.	.408	.461	.515	.603	.692	.442	.452	.591	.662	.737	.444	.493	.593	.726	.821
Average per Sq. Ft. Steel Lbs.	2.92	3.28	4.00	5.55	5.54	3.25	4.03	4.29	5.79	6.21	3.67	3.94	4.43	5.63	6.23
Average per Sq. Ft. Form Lumber	3.74	3.75	3.84	3.95	3.95	3.50	3.63	3.82	3.83	3.86	3.72	3.75	3.87	4.02	4.12

The girders for 20′x40′ and 20′x50′ were assumed as freely supported.

TYPICAL FLOOR CONSTRUCTION.

Total Load per Sq. Ft.	125	150	200	300	350	125	150	200	300	350	125	150	200	300	350
Column Centers	22′x28′–Type F,–Sl. Slabs–13 Spts–18′C.					22′x30′–Type F,–Sl. Slabs–16 Spts–20′C.					24′x24′–Type F,–Sl. Slabs–13 Spts–18′C.				
Number of Girder	66	68	76	84	85	64	69	82	85	90	58	62	65	82	93
Number of Beam	24	26	41	52	51	31	39	42	45	53	24	26	41	49	51
Thickness of Slab	4	4½	5	5½	6	4	4½	5	6	6	3½	3½	4½	5	5½
Weight of Steel in Slab	.69	.75	.85	1.70	1.75	.91	1.01	1.27	1.70	2.25	.59	.90	.75	1.17	1.23
Average per Sq. Ft. Concrete C. Ft.	.500	.560	.652	.767	.809	.490	.555	.653	.784	.873	.444	.472	.584	.685	.875
Average per Sq. Ft. Steel Lbs.	3.36	4.10	4.50	6.38	7.12	3.90	4.03	5.10	7.20	7.31	3.36	4.25	5.13	5.79	7.89
Average per Sq. Ft. Form Lumber	3.62	3.65	3.87	4.07	4.07	4.07	4.07	4.14	4.23	4.44	3.64	3.70	3.91	3.96	4.20
Column Centers	24′x26′–Type F,–Sl. Slabs–16 Spts–18′ C.					24′x28′–Type F,–Sl. Slabs–16 Spts–20′C.					24′x30′–Type F,–Sl. Slabs–16 Spts–20′C.				
Number of Girder	62	63	70	83	84	63	65	71	84	85	65	70	82	85	86
Number of Beam	31	40	42	45	53	32	40	47	51	54	32	41	43	53	54
Thickness of Slab	3½	4	4½	5½	6	4	4½	5	5½	6	4	4½	5	6	6
Weight of Steel in Slab	.79	.70	1.02	1.25	1.33	.69	.75	.85	1.70	1.75	.91	1.01	1.27	1.70	2.25
Average per Sq. Ft. Concrete C. Ft.	.454	.532	.601	.726	.809	.515	.560	.648	.794	.796	.503	.584	.642	.786	.786
Average per Sq. Ft. Steel Lbs.	3.67	3.84	4.86	6.54	6.65	3.70	4.04	4.54	7.43	7.57	4.07	4.39	5.24	7.00	8.23
Average per Sq. Ft. Form Lumbnr	3.81	3.91	3.92	3.92	4.06	3.97	4.05	4.05	4.18	4.18	3.86	3.86	3.90	4.06	4.06

DIMENSIONS AND PROPERTIES OF GIRDERLESS FLOOR CONSTRUCTIONS.

BAY DISTANCES IN FEET

	12'				13'				14'			
Total Load per Square Foot in Pounds	Thickness of Slab in Inches	Area of Reinforcement per Foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips	Thickness of Slab in Inches	Area of Reinforcement per Foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips	Thickness of Slab in Inches	Area of Reinforcement per foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips
100	4	.12	.85		4	.15	1.07		4½	.14	1.00	
150	4½	.165	1.20		5	.15	1.07		5	.21	1.50	
200	5	.200	1.42		5½	.20	1.42		6	.22	1.56	
250	5½	.235	1.68	2′–9″	6	.25	1.78	3′–0″	6½	.27	1.92	3′–3″
300	6	.250	1.78		6½	.27	1.92		6½	.37	2.65	
350	6½	.270	1.92	4′–3″	6½	.36	2.56	4′–6″	7	.42	3.00	5′–0″
400	7	.290	2.06		7	.39	2.78		7½	.42	3.00	
450	7½	.310	2.20		7½	.41	2.92		7½	.54	3.85	
500	7½	.365	2.60		7½	.50	3.55		7½	.63	4.50	

BAY DISTANCES IN FEET

	15'				16'				17'			
100	5	.15	1.07		5	.16	1.14		5½	.17	1.21	
150	5½	.20	1.42		6	.20	1.42		6	.26	1.85	
200	6	.28	2.00		6½	.27	1.92		6½	.35	2.50	
250	6½	.35	2.50	3′–6″	7	.36	2.56	3′–8″	7	.44	3.13	4′
300	7	.38	2.70		7	.50	3.55		7	.60	4.26	
350	7½	.45	3.20	5′–3″	7½	.54	3.84	5′–6″	7½	.65	4.62	6′
400	7½	.54	3.85		7½	.67	4.76		8	.70	5.00	
450	7½	.63	4.50		8	.69	4.90		8	.85	6.05	
500	8	.67	4.76		8	.82	5.82		9	.76	5.40	

DIMENSIONS AND PROPERTIES OF GIRDERLESS FLOOR CONSTRUCTIONS.

BAY DISTANCES IN FEET

	18'				19'				20'			
Total Load per Square Foot in Pounds	Thickness of Slab in Inches	Area of Reinforcement per Foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips	Thickness of Slab in Inches	Area of Reinforcement per Foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips	Thickness of Slab in Inches	Area of Reinforcement per Foot of Strip	Weight of Reinforcement per Sq. Ft.	Diameter of Capital Width of Strips
100	5½	.18	1.3	4'–2"	6	.19	1.40	4'–4"	6	.25	1.77	4'–6"
150	6½	.25	1.8	6'–4"	6½	.32	2.30	6'–8"	7	.32	2.27	7'–0"
200	7	.37	2.7		7	.44	3.20		7½	.45	3.20	
250	7½	.45	3.2		7½	.54	3.90		7½	.65	4.60	
300	7½	.63	4.5		8	.62	4.40		8	.75	5.30	
350	8	.67	4.8		8	.82	5.80		9	.75	5.30	
400	8	.83	5.9		9	.80	5.70		9	.92	6.50	
450	9	.80	5.7		9	.92	6.60		10	.86	6.10	
500	9	.91	6.5		10	.86	6.10		10	1.1	7.80	

BAY DISTANCES IN FEET

	21'				22'				25'			
100	6	.27	1.91	4'–9"	6½	.25	1.78	5'–3"	7	.33	2.35	5'–9"
150	7	.38	2.70	7'–4"	7	.46	3.26	7'–8"	7½	.59	4.20	8'–9"
200	7½	.54	3.80		7½	.60	4.25		8	.80	5.68	
250	8	.67	4.80		8	.76	5.40		9	.87	6.20	
300	9	.67	4.80		9	.80	5.70		10	.90	6.40	
350	9	.86	6.10		10	.78	5.52		10	1.10	7.80	
400	10	.84	6.00		10	.96	6.80		10	1.40	9.90	
450	10	1.00	7.10		10	1.20	8.50		11	1.45	10.30	
500	10	1.20	8.50		10	1.30	9.20		11	1.58	11.20	

EXPLANATION OF TEE BEAM TABLES AND RULES FOR THE DESIGN AND CONSTRUCTION OF REINFORCED CONCRETE BEAMS.

The seemingly irregular sizes of the stems of the Tee are adopted for the reason that in most of the localities of the United States the commercial sizes of lumber are less than the nominal sizes. A so-called 2"x10" plank is often only 1⅝"x9⅝", and if the two edges of the plank are sized these dimensions may be still less. This is the reason why beams No. 1 to No. 98 have a width less than an even number of inches, while beams No. 201 to No. 265 may be used where lumber in full sizes is obtainable. As stem depth we adopted the greatest depth which can be formed by the commercial sizes of the planks, used for the side of the forms, without any waste. This can be more clearly seen from inspecting the table on beam forms.

The safe bending moment, which may be allowed for these beams, were found by the approximate, but safe and convenient rule: Moment in foot-pounds=18000×area in square inches of the reinforcement × depth of stem in feet, which stem depth we substitute for the distance of center of reinforcement to center of compression. The fibre stress of 18000 pounds is about ⅓ of the elastic limit of commercial high carbon bars or cold twisted steel bars.

This rule supposes that the entire compression is taken up by the flange or slab, which compression must = the tension in the steel rods. Allowing a fibre stress of 400 pounds per square inch in compression in the slab, we find that the least available width of the flange must = 45 × area of reinforcement in square inches divided by the thickness of the slab in inches.

This width is, however, also governed by the consideration that it cannot be greater than the distance between the beams, nor greater than 400 × area of the reinforcement, divided by the total depth of stem and flange in inches, or, in other words, the reinforcement of the ideal slab cannot be less than ¼%. It is generally sufficient to check the strength of the flange of the Tee beam by the first rule.

In figuring the shear of the beams we assumed that the thickness of the slab is four inches for all beams having a stem 12¾″ deep or less, and five inches for deeper beams, and that the allowable shear per square inch of the cross section may be 60 pounds without any special provision for shear. This does not mean, however, that stirrups may be dispensed with. The number of stirrups, even in this case, should not be less than the number of lineal feet in the span. These stirrups should be spaced 6″ c. c. near the supports and 18″ in the center of the beam, and serve mostly the purpose of preventing shrinkage cracks at the junction of stem and flange. It is not advisable to use beams where the shear exceeds 125 pounds per square inch of the concrete section. although quite a number of beams with a larger shear have done service for many years. Where the shear exceeds 60 pounds per square inch, the bending up of part of the number of the rods is absolutely necessary, and the number of stirrups must be increased. It is not advisable to increase the number to more than 1½ times the number above mentioned, and it is preferable to increase the size of the stirrup bars. The spacing of the stirrups may be found by the following rule: Substract from the actual shear the shear given in the beam tables and multiply the depth of the stem in inches by 10000 times the sectional area of the stirrups and divide by one-half of the above mentioned difference of shear values. Experiments prove that the bending up of the reinforcing bars in different adjoining planes increases the strength of the beam in shear to a very considerable extent and that we are justified in assuming that the bent-up portions of the bars take up at least one-half of the shear, which is not taken up by the concrete, and that the other half is taken care of by the stirrups.

Where beams are freely supported and uniformly loaded, the bending moment is found by the formula, $\frac{pl^2}{8}$ when $p =$ total load per lineal foot, and $l =$ span in feet, which is somewhat larger than the clear span. For continuous beams

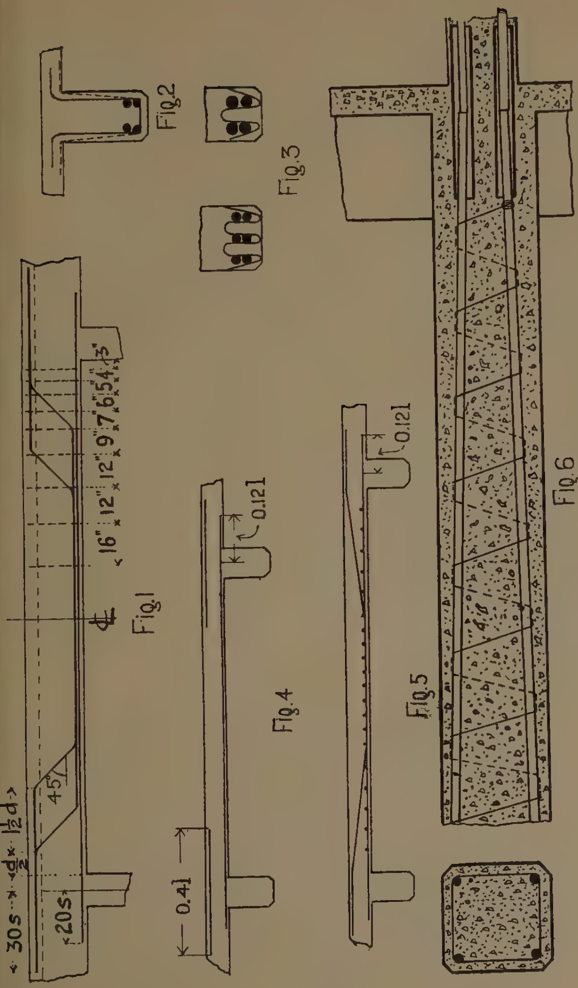


Fig. 1

Fig. 2

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Typical Girder, Slab and Column Design.

on three or more supports, the bending moment at the center may be taken as $\frac{pl^2}{10}$, and can be conveniently found in tables on pages 22 to 25.

Large negative bending moments appear in continuous beams over the supports, producing a reversal of stresses at these points. While present practice has fully decided to take care of the full negative moments in case of continuous slabs, by placing the same amount of reinforcing on top of the slabs over the supports as at the underside in the center, the practice in regard to continuous beams is yet undecided. This may be ascribed to the fact that signs of weakness of continuous Tee beams at the supports are very rare and may be explained by the assumption that the concrete slabs take up a great portion of the tensile stresses.

The short brackets forming an angle of 60 degrees or more with the horizontal, as frequently found in practice, are mostly adopted for good luck and without any statical consideration; their adoption considerably increases the cost of the structure and simply shifts the weak point a few inches away from the columns and, besides, tends to increase these negative moments. The practice of to-day has the tendency to suppress the brackets entirely or to make them very flat. Moreover, a charge of $1 to $1.50 is made by plasterers for plastering each bracket, and they often interfere with partitions.

The tensile stresses over the supports are generally only taken care of by extending the bent-up bars more or less into the adjoining beam. In very heavy beams, however, and especially bridge girders, extra rods of the same section as the straight rods and of a length = 0.4 of the span should be imbedded at the top over the supports.

Figure No. 1 shows a standard design of a beam. Half the number of the rods are straight and of a length $= L + 40s$, when $L =$ span c. c. supports, and s the diameter of the rods, and the other half are bent up in various planes and of a length $= L + 60s + d$, when $d =$ the depth of the stem.

The bends of the rods should never be sharp and the radius of the bend not less than 1½ s, which allows of cold bending of the rods, even for the largest diameters. It is preferred practice to arrange the rods in pairs in vertical planes, and the writer's practice to keep them in their proper places is shown in Figures Nos. 2 and 3. The stirrups, which in most cases are made of ¼ to ⅜″ round or square wire, are originally made in the shape shown in dotted lines. They are forced into the beam form and rest with their horizontal legs on the floor boards. They possess considerable spring, which produces a friction against the sides of the forms, preventing any shifting in a horizontal direction. They carry the weight of the rods and prevent them from approaching nearer to the underside of the beam than is desired. Two or three bent spacing wires, as shown in Figure No. 3, keep the rods in place in transverse direction. After the beams are concreted, the stirrups are pushed back, by means of tamping bars, into the position shown in black in Figure 2. The writer considers it essential that the stirrups reach well into the slab, as otherwise a separation of stem and flange can hardly be prevented sooner or later, especially in case of fire. The use of small hooks at the end of girder rods increases the cost without any adequate increase of strength.

It is good practice to give the beams a camber of at least ½″, or 1/300 of the span.

Example: A continuous beam, 18′ c. c. supports, which supports are 12″ wide, carries a slab 10′ wide with a total floor load of 250 pounds per square foot. The total load per lineal foot of beam = 2500 pounds, and the bending moment $\frac{2500 \text{x} 17^2}{10} = 72000$ foot-pounds, which also may be found from tables on page 24 by multiplying the bending moment for 250 pounds by 10. On page 7 we find that beam No. 40 will answer. The total shear at the supports $= 2500 \times 8.5 = 21250$ and at a point 2′ 6″ away from the supports $= 2500 \times 6 = 15000$. Assuming that we use ¼″ round wire for the stirrups, the area of both legs of the stirrup is 0.098 and the spacing at both points is given by $\frac{10,000\text{x}0.098\text{x}16.25}{4550} = 3.5$ and $\frac{10,000\text{x}0.98\text{x}16.25}{1400} =$ 11.0″, where 4550 and 1450 = half the difference between the actual shear and the shear of 12100 as given in the beam table for No. 40. Figure No. 1 shows the distribution of stirrups which may be safely used. The cost of a stirrup is only a cent or two in most cases, and it will generally suffice to figure the shear at the supports only and to determine the spacing at this point, and to increase the spacing successively by one inch until one-quarter of the span is reached.

The following table will facilitate this work:

Let d = the stem depth of the beam, and c = spacing of the stirrups; then for the various relations of d:c and for the various sizes of stirrup bars, the allowable difference between the actual shear and the shear given in the beam tables are

$\frac{d}{c} =$	10	9	8	7	6	5	4	3	2	1
$\frac{1}{4}$″ **Round**	19600	17500	15700	13700	11800	9800	7800	5000	3900	1960
$\frac{1}{4}$″ **Square**.....	24500	21900	19800	17200	14800	12300	9750	7400	4900	2450
No. 0 Wire	29400	26300	23600	20600	17700	14700	11700	8900	5890	2950
$\frac{3}{8}$″ **Wire** ...	44300	39500	35000	31600	26600	22100	17600	13300	8800	4200

Second Example: A continuous beam, forming part of a square slab, 18′x18′, is supported on columns 12″ square. The total load per square foot on the slab is 250 pounds, and this slab, on account of being reinforced in both directions, will transmit to each beam ¼ of the panel load. This load will be in form of a triangular load, having its apex in the center of the beam. If there are panels on both sides the load per lineal foot in the center = $18 \times 250 = 4500$ pounds, and becomes zero at the supports. The bending moment in such a case is ⅔ of the moment produced by a uniform load of 4500 pounds per lineal foot, or $= \frac{2}{3} \times \frac{4500 \times 17^2}{10} =$ 87000 foot-pounds, which closely corresponds to beam No. 41, or to beam No. 39 when square bars are used.

It will be noticed that in both cases only the clear span was used in the calculation. More conservative designers may possibly use a mean between the clear span and the span center to center of supports, but it is a clear waste of money to figure with the span center to center of supports, when the latter are two feet wide or wider. Only in the case of beams which are carried by girders the writer advocates to figure with the span center to center of supports. In case of non-continuous beams the span is to be taken as center to center of bearing.

For the convenience of persons of little experience in reinforced concrete construction, who wish to share in the alleged enormous profits made in this line of construction, quite a number of unit systems of ready-made and assembled beam reinforcement are at present in the market. While they may be a good thing for inexperienced persons, they are decidedly more expensive than the ordinary method above described. It is the writer's practice to have on every job, besides competent foremen, an experienced engineer as superintendent, to watch the distribution of the steel; and this is a far better safeguard than unit reinforcements, which in the hands of incompetent men, do not prevent of placing the units in beam forms, where they do not belong.

EXPLANATION OF SLAB TABLES.

Experiments have fully demonstrated the fact that the saf bending moment in foot-pounds per lineal foot of a slab c the thickness t in inches is equal to coefficient x t^2. Thi coefficient depends on the amount of reinforcing per linea foot, the mixture of the concrete and the distance of th center of the reinforcing bars from the compression face o the slab. We assume that the latter distance amounts t 0.87 t, or that the center of the bars is 0.13 t from the tensio side. Where it is desired that the latter distance is greate a corresponding addition is to be made to the thickness a found in the tables.

The amount of reinforcing per lineal foot is often e pressed in percentages of the steel area to the concrete are We say that a four-inch slab is reinforced by 1% of stee when the area of the steel bars per lineal foot of sla $= \frac{4x12}{100} = 0.48$ square inches, or that a five-inch slab is re inforced by .35%, when the steel area per lineal foc $= \frac{5x12}{100} \times 0.35 = 0.21$ square inches.

The following table gives the values of the coefficient a found by many hundred experiments and by innumerabl application in practice:

Percentage	Coefficient	Mixture	Percentage	Coefficient	Mixture	Percentage	Coefficient	Mix ture
0.25	50	1:6	0.85	98	1:6	1.45	146	1:5
0.30	54	1:6	0.90	102	1:6	1.50	150	1:5
0.35	58	1:6	0.95	106	1:6	1.55	154	1:4
0.40	62	1:6	1.00	110	1:6	1.60	158	1:4
0.45	66	1:6	1.05	114	1:6	1.65	162	1:4
0.50	70	1:6	1.10	118	1:6	1.70	166	1:4
0.55	74	1:6	1.15	122	1:6	1.75	170	1:4
0.60	78	1:6	1.20	126	1:6	1.80	174	1:4
0.65	82	1:6	1.25	130	1:6	1.85	178	1:4
0.70	86	1:6	1.30	134	1:5	1.90	182	1:4
0.75	90	1:6	1.35	138	1:5	1.95	186	1:4
0.80	94	1:6	1.40	142	1:5	2.00	190	1:4

Slabs, reinforced with less than ¼%, act as plain concrete slabs.

The percentages greater than one are seldom used.

The mixtures 1:6, 1:5, and 1:4 mean concrete mixtures of one bag of cement, weighing at least 94 pounds, to six, five and four cubic feet, respectively, of sand and gravel or crushed rock. While it is usual to give the volumes for sand and gravel or rock separately, experienced concrete men will agree that the relation of sand to gravel or rock has often to be varied on the same job in order to obtain the densest concrete or the best results. The reason is that the voids in the stone are fairly uniform, while those in the sand vary from 30 to 60%.

The tables on pages 22 to 25 give the bending moments for various total loads per square foot and various spans for continuous slabs over three or more supports, which slabs we call simple slabs. The tables on pages 26 to 28 give the bending moments for continuous slabs, supported on four sides, which, for convenience, we call square slabs.

All these bending moments are for a slab ONE FOOT WIDE, and throughout this book the slabs are figured by first finding the bending moment per lineal foot width of slab.

As mentioned in the explanation of reinforced concrete ams, in continuous slabs appear negative moments over the pports, which should be taken care of by the same number rods as placed at the underside of the slabs. Formerly it was tried to accomplish this by bending up over the supports all or half the number of rods. While in theory this looks simple enough, it is a difficult thing to obtain in practice. The rods have a tendency to turn sideways, and very often there is hardly a rod in five feet which is actually near the top face over the supports. The result is that cracks appear in the slabs at, or a few inches alongside, the beams. This is the reason why the writer advocates placing separate rods of the same number as those used at the underside and of a length = 0.4 of the span, at the top of the slab. These rods placed after the slab is concreted and are pressed into the wet concrete. The writer does not find the same difficulty in case of square slabs, for the reason that the spans are much larger and the rods, if temporarily supported by short pieces of scantling or permanently by special bars, will hang through in a catenary and have no tendency to turn sideways. Therefore, no extra rods are to be placed in square slabs.

Figures 4 and 5 show the standard design for simple and square slabs. For the bending moment per lineal foot of square slabs from the total dead and live load we adopt the formula $\frac{pl^2}{24}$ in case of continuous slabs, and $\frac{pl^2}{20}$ in case of freely supported slabs.

This represents the average bending moment per lineal foot in each of the two main directions. A simple consideration will show that the maximum bending moment must occur in the central portion of the slab, and for this reason we propose the following rule for the spacing of bars in square slabs.

After having determined the sectional area of the reinforcing per lineal foot and the corresponding sizes and spacing of the bars from tables on pages 12 to 15, divide the span in inches by the spacing of the rods in inches and obtain the total number of the rods, required in one direction. Place one-half of this number in the middle third of the span, while the other half may be equally distributed in the outer thirds.

Where panels are not exactly square but where the ratio of the sides is not greater than 1.33, we find the average moment per lineal foot by the formula $\frac{pl^2}{24}$, when $l =$ mean between the two sides in feet.

It is not economical to consider slabs supported on four sides when the ratio of the sides is greater than 1.5.

In some instances, as for dead loads, continuous water and earth pressures, continuous footings, it is good practice to figure the bending moment in continuous simple slabs by the formula $\frac{pl^2}{12}$

Examples: A continuous slab has to support, on a clear span of 9′ 6″, a total load of 350 pounds per square foot. The bending moment per lineal foot $= \frac{350 \times 9.5^2}{10} = 3150$ foot-pounds, which can also readily be found in table on page 23. On page 17 we find that a 6″ slab with 0.504, or a 6½″ slab with 0.42 square inches of reinforcement per lineal foot will answer. If we adopt the first, we can find on page 14 that ½″ square bars, 6″ c. c. will suffice. Over the supports we shall place the same rods, but only 4′ long. The longitudinal reinforcement should not be less than ⅛% = 0.09 square inches per lineal foot, which corresponds to ⅜″ square bars, 18″ c. c. The weight of a bar of steel one inch square and one foot long is 3.4 pounds. Hence the theoretical weight of the reinforcement per square foot is found by multiplying the area per lineal foot by 3.4. On account of the rods extending at least 0.12 of the span beyond the center of the beams and on account of the extra rods, this coefficient increases to an average of 5.3, and the weight of the reinforcement per square foot in our case $= 0.504 \times 5.3 = 2.7$ pounds, which is also given in table on page 14. To this is to be added the weight of the longitudinal reinforcement for which the coefficient of 4 will suffice, or can be found on page 13.

Second Example: A continuous square slab of 15.5′ clear span has to support a total load of 150 pounds per square foot. The average bending moment per lineal foot $= \frac{150 \times 15.5^2}{24} = 1500$ foot-pounds, which can also be taken directly from table on page 28. On page 17 we find that a 5″ slab, reinforced by 0.21 square inches, will do. On page 12 we learn that 0.21 corresponds to ⅜″ round bars 6¼″ c. c. This requires 30 bars in the length of 15.5′, 15 of which we we shall space 4″ c. c. in the middle third, while in each outer third we shall have seven spacings of 7¼″.

We can use the same thickness of slab and the same reinforcement if the slab has the dimensions 14′x17′, with the exception that 33 rods will run in the shorter way, and 27 rods will run the long way. It is the writer's practice to let only those rods hang through a catenary, which are placed in the middle third, while those in the outer third remain straight. This considerably facilitates the placing of the rods. The wiring of floor rods at each intersection is a great waste of money. Two at the utmost three points in each rod need only to be fastened to other rods. The binding wire should be No. 16 annealed wire. The rods should extend at least 0.12 of the span beyond the center of the beams, both in simple and square slabs.

Third Example: A lintel, 17″ wide and of a clear span of 9′, has to support 2000 pounds per lineal foot. Assuming the bearing is 9″, we shall take the span =9.75′. The bending moment $= \frac{2000 \times 9.75^2}{8}$ =24000 foot-pounds, or per lineal foot width of lintel 24000÷1.42=17000 foot-pounds. On page 19 we find that this requires a slab 15″ thick, reinforced by .99 square inches per lineal foot width, or 1.41 square inches for a width of 17″, which corresponds to 2-⅝″and 2-¾″ round bars, as can be seen on page 5.

It may be well before closing this chapter to remark that the longitudinal reinforcement of ⅛% is in fact too small to prevent cracks. It represents, however, actual practice, and no great inconveniences occurred. In roofs and other structures exposed to the elements, the longitudinal reinforcement should not be less than ¼%. The weight of a slab per square foot, in pounds, can be found by multiplying the thickness in inches by 12.

EXPLANATION OF TABLES ON WALL AND COLUMN FOOTINGS.

The bending moment in foot-pounds in wall footings of the type shown in Figure No. 9, we assume to be equal to $\frac{pl^2}{2}$ when p the load on the ground per square foot and l the projection in feet. Having found the bending moment, we can find the thickness of the concrete and the area of the reinforcing from the slab tables on pages 16 to 20. The shear in footings is greater than in any other member of construction, and therefore we recommend and have adopted the minimum percentage of reinforcement of ¼%, to obtain the greatest depth of footing. We recommend the use of comparatively small bars with close spacing for the reinforcement. For example: where 0.6 square inches of steel per lineal foot is required, the use of ½″ square bars 5″ c. c. will be preferable to ⅝″ square bars 7¾″ on centers.

We consider column footings as cantilevers in four directions, and the theoretical bending moment at a cross section through the center of the footing would be $\frac{ps^2}{12}$, if the column load be concentrated at a point in the center. In as much as the column distributes the load over its own area, the bending moment is reduced in the footings shown in table to about $\frac{ps^2}{15}$, when p the load on ground per square foot, and s the side of footing in feet. On account of the trapezoidal form of the section, we consider the footing as a slab of only half the width of the footing. On account of the great shear values we again adopt only the minimum amount of reinforcement of ¼% in order to obtain the greatest depth. Also here the use of small rods and close spacing is recommended.

For Example: A footing has to support a load of 180,000 pounds on ground which will carry 6000 pounds per square foot. On page 32 we find that the footing should be 5′ 6″ square. The depth of the footing in the center is 22″ and at

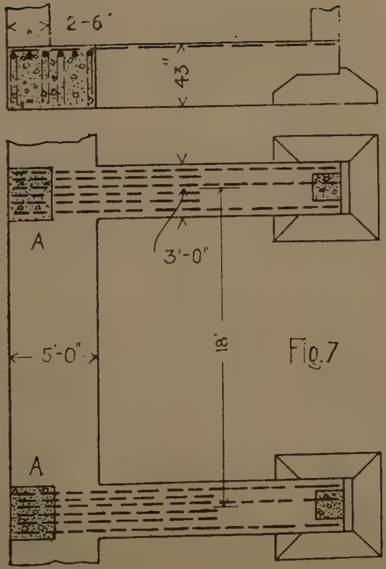


Fig. 7

Cantilever Footings.

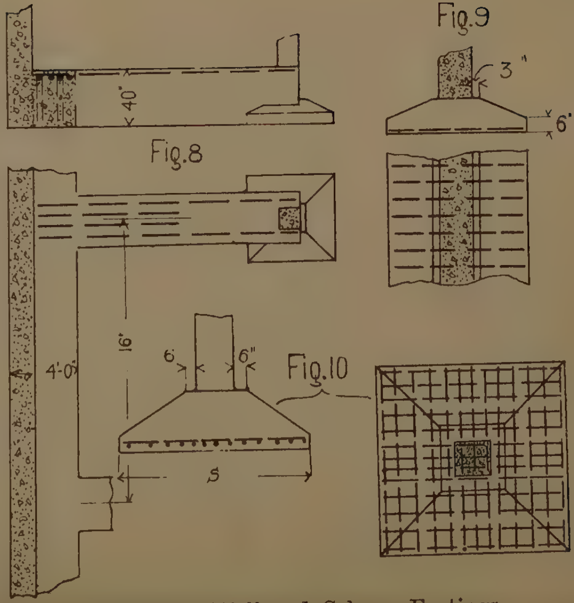


Fig. 8

Fig. 9

Fig. 10

Standard Wall and Column Footings.

the edge 8″. We have to place 1.82 square inches of steel in the two main directions, which correspond to nine ½″ round bars in each direction. No column smaller than 14″ should be placed on this footing, and in figuring the cubical contents it was assumed that the top is 14+12=26″ square. Such a footing would contain 37 cubic feet of concrete and 70 pounds of steel.

Cantilever Footings: Where a column or wall footing must be kept inside the property line and where the width of the footing is so great that the center of pressure comes outside of the middle third of the footing and produces too high a unit pressure on the ground, a cantilever construction, as shown in Figures Nos. 7 and 8 should be resorted to.

For an example we shall assume that the column A has to support 450,000 pounds and that the allowable pressure on the soil is 5000 pounds per square foot. Columns A are 18′ on centers; this requires a footing alongside the property line $\frac{450.000}{5000\text{x}18}$ =5′ 0″ wide. This footing we figure as a slab of a clear span of 18′—3′ 0″=15′ and loaded by 5000 pounds per square foot. The bending moment in this slab $\frac{5000\text{x}15^2}{12}$ =94000 foot-pounds per lineal foot of width. The column can take up the load from 2′ 6″ width of the slab as a symmetrical load, but the load from the other 2′ 6″ must be transferred to the column by means of the cross-beam "AB." We err on the safe side if we assume the bending moment on the cross-beam to be $5000\times2.50\times18\times\frac{2.50}{2}$ =280,000 foot-pounds. This assumes that the cross girder does not act as footing itself, and in building the cross girder loose rock one foot deep should be placed at the bottom to allow of slight settlements.

The formula: bending moment in foot-pounds=Coefficient ×square of thickness of slab in inches, will enable us to find the dimensions of the footing slab and cross girder. For ¼% of reinforcement the coefficient = 50 or the square of the thickness for the footing slab = $\frac{94.000}{50}$ = 1880, or the thickness 43″. The bending moment per foot width of cross girder=280,000÷3=93300 foot-pounds; therefore, a slab of 43″, reinforced by ¼%, will also answer.

The amount of steel in the footing slab $= 60\times 43\times\frac{1}{400}=6.5$ square inches, and in the cross girder $= 36\times 43\times\frac{1}{400}=3.8$ square inches, corresponding to three 1⅛″ and three 1¼″, and to three ⅞″ and three 1″ bars, respectively. It is clear that these rods must be placed at the top of the slabs, and in case of the footing slab, the three 1¼″ bars should be bent down at the supports to take care of the shear and of the negative moments. Besides, three 1⅛″ bars, 7′ 3″ long, should be placed at the bottom of the footing slab at the columns, also to take care of the negative moments. Figuring the cost of concrete at 30c per cubic foot, steel at 2½c per pound, the average cost of this cantilever footing per lineal foot of wall is only $9.20.

Second Example: Let us assume a 17″ concrete basement wall has to support 20000 pounds per lineal foot. The interior columns are spaced 16′ on centers. Allowing 5000 pounds per square foot on the ground, the width of the wall footing =20000÷5000=4′. The width of 4′—1′-5″=2′-7″ produces an excentric loading, and must be designed to transmit the load to the cross girders. The bending moment $= \frac{5000 \text{x} 13.5^2}{12}$ =76000 foot-pounds. For ¼% of reinforcement the square of the depth in inches=76000÷50=1530, or the depth practically 40″, and the amount of reinforcing $=33\times 40\times\frac{1}{400}=$ 3.1 square inches, or four 1″ round bars.

The bending moment for the cross girder $= 5000\times 2.583\times 16\text{x}\frac{2.583}{2}$ =267,000 foot-pounds, or per lineal foot width= 89000. A depth of 40″ requires a coefficient of 89000:1600= 56, or per table on page 64 a percentage of 0.325. The area of the steel in the cross girder $=\frac{36 \text{x} 40}{100}\times 0.325=4.7$ square inches.

The cost of this footing is only $7.49 per lineal foot of wall. Careless designers often omit to take care of the excentric loading, and the whole saving amounts only to $3.50 per lineal foot in such a case as this, but endangers the stability of the building.

EXPLANATION OF THE COLUMN TABLES.

The sizes of the columns here adopted are governed by the commercial sizes of lumber, and the scheme of the form work may be found on page 102. Most of the building ordinances of the larger cities of the United States allow a stress of 450 pounds per square inch on the concrete section, and 15 times this amount = 6750 pounds per square inch on the steel section, which specifications represent safe practice, but should be amended in regard to the limiting of the amount of reinforcing and in regard to the mixtures of the concrete. Experiments have demonstrated that the steel does not take 6750 pounds stress if the reinforcement exceeds 5% of the cross section; they have further demonstrated that we can increase the stresses both on the concrete and on the steel, if we use richer mixtures than one part cement to six parts of sand and crushed rock, and we can lay down the following rules:

For 1:6 mixture allow per square inch 450 pounds on concrete and 6750 pounds on steel;

For 1:5 mixture allow per square inch 500 pounds on concrete and 7500 pounds on steel;

For 1:4 mixture allow per square inch 550 pounds on concrete and 8250 pounds on steel;

For 1:3 mixture allow per square inch 600 pounds on concrete and 9000 pounds on steel;

which increases the loads in the tables 11%, 22% and 33%, respectively; or we have to decrease the given column loads by 9%, 18% or 25% in order to use readily the table figures.

It is cheaper to use richer mixtures, but at the same time it must be considered that richer mixtures are less fireproof than leaner mixtures. Most building ordinances specify that the outer inch of concrete shall not be considered as carrying stress, and in this case take the column one size larger than found in the tables.

The value of ties binding the column rods together is largely over-estimated. The ordinances generally specify the ties to be not farther apart than the least dimension of the column, without specifying the size of the ties or the kind of connections. We to-day know that the steel rods have only the task of keeping the strength of the concrete within the same value as that found in compression tests on cubes of 12″ size, and that the failure of columns are produced by shearing of the concrete on 45°, which failure cannot be prevented by the ties ordinarily used. Ties are, however, necessary in order to hold the steel rods in place, and in some cases to guard against shear, especially in excentrically loaded columns.

The writer finds it cheaper to use coils instead of loose ties, which coils consist of No. 3 soft annealed wire, and have a pitch of 12″. They are made by winding the wire on a collapsible core. The weight in pounds per foot of these coils is found by dividing the side of the coil in inches by 20. Where columns connect with smaller columns on top it is the writer's practice to slope the rods uniformly from the bottom to the top where the difference in size is not very considerable, say not more than eight inches. In this case, the coils must reduce in diameter towards the top, which is done on a tapered core.

The column rods should be connected at each floor level, either by over-lapping of the rods or by means of gas pipe sleeves, as given in the tables. The space between the steel rods and the pipes should be carefully grouted with cement and sand in proportion of one to two. Where the size of the columns as given in pages 36 to 41 is too large, Considere columns or hooped columns, as given on pages 42, 43, may be used. Considere demonstrated that hooping increases the strength of the concrete 20%, and that the value of the reinforcing by coils or hoops is 2.4 times the value of the reinforcing by longitudinal rods, provided that the pitch of the helix of the coils or the distance of the hoops does not exceed 1/7 of the diameter of the coils. Assuming 1:3 concrete, we are justified to allow 720 pounds per square inch in compression on the concrete, 10800 pounds per square inch on the longitudinal reinforcement, and 25420 pounds per square inch on the ideal longitudinal reinforcement which has the same weight as the coils adopted.

Where even Considere columns give too large dimensions for the columns, structural steel columns enclosed by concrete should be adopted. Allow 12000 pounds fibre stress on the steel section, and 300 pounds per square inch on the concrete section.

The Column Tables give the load in 1000 pounds for columns reinforced by round bars.

The milling of the ends of rods at the column connections is considered an expense wrongly applied. It is nearly impossible to bring the rods to a square bearing, and in as much as the rods only safeguard the cubical compressive strength of the concrete, the gas pipe sleeves amply fulfill all requirements for transmitting occasional tensile stresses.

The connection of the columns with the footings is best done in the following manner: When concreting the footings, insert tapered cores about one inch larger in diameter than the size of the column rods and of a length to reach within 4″ to 6″ of the bottom of the footing. Of course, the position and number of these cores must correspond with the column rods. The cores must be pulled out within six to twelve hours after concreting, and the holes temporarily closed by wooden stoppers to prevent dirt falling in. After the column rods are inserted fill in with grout of a mixture one part cement to one of sand.

EXPLANATION OF TABLES ON FLOOR CONSTRUCTION ON PAGES 44 TO 54.

In order to readily estimate the cost of reinforced floor and girder construction, to compare the cost of floors of different spans and floor loads, and to relieve the busy engineer and architect of a great deal of mechanical work, these tables were figured for the spans most commonly used in construction work. The figures at the top mean the distances of the columns center to center in feet; for example: 12/16 means that the columns are spaced 12′ in one direction and 16′ feet in the other direction. Only in loft, factory or warehouse buildings is the choice of the arrangement or girders and beams entirely free. In other buildings, it is often desired that the beams coincide with certain partitions, or that the floor construction offers a pleasing appearance, which latter effect is generally obtained by square panel constructions. It is obvious that the cost of the floor and beam construction will depend on the amount of concrete, steel and form lumber required, and the best basis of comparison is the average cost per square foot, and for this reason we give in the tables the average quantities of concrete, steel and form lumber per square foot, which includes girders, beams and floor slabs. In most of the cases the cheapest possible arrangement of beams and girders was adopted, although a change in the number of direction of beams may vary the cost only a few per cent. The fact that the actual amount of material required for a certain floor construction is less than that required in another does not always mean that this floor construction is actually the cheapest. If in this floor construction more beams are used than in another, it is to be considered that the unit labor for the form work is increased, also that of the steel labor and to a smaller extent that of the concrete labor.

Figures 11-17 show that we only adopted seven different types of girder, beam and slab arrangements for the floor constructions on pages 44-54. In these tables the first two lines give the serial number of the girder or beam of the beam tables; the third line gives the thickness of the floor slab; the fourth line, the weight of steel per square foot in the slab (except the weight of longitudinal rods in simple slabs), by which weights the proper reinforcement may be found from tables on pages 12 to 15; the fifth line gives the average amount of concrete in cubic feet per square foot of floor, including all girders, beams and slabs; the sixth line gives the average weight of steel of the floor construction; and the seventh line, the average amount of lumber required for the forms of girders, beams and slabs for the story heights noted at the head.

The cost of a cubic foot of concrete in place varies from 20 to 30c; the cost of a pound of steel in place from two to three cents; the cost of the forms from $40 to $50 per 1000 feet B. M., acording to localities, with a very close average of the mean of these figures for most of the large cities of the United States, or 25c per cubic foot of concrete, 2½c per pound of steel, and $45 per 1000 feet B. M. for the forms. These figures, as a rule, do not include the contractors' profit or installation of plant, office expense, etc.

For heavy loads, say 250 pounds per square foot and over, bays 14'x14' are generally the cheapest. According to our tables the cost per square foot of a floor 14'x14' and 250 pounds per square foot total load $= 0.58 \times 25c + 3.45 \times 2\frac{1}{2}c + 2.95 \times 4.5 = 36.7c$ per square foot. For bays 16'x16' the cost per square ft. would be $0.594 \times 25c + 4.25 \times 2\frac{1}{2}c + 3.13 \times 4.5 = 40.4c$; and for bays 18'x18' and the same floor load the cost per square foot would be $0.702 \times 25c + 4.63 \times 2\frac{1}{2}c + 3.14 \times 4.5 = 43.4c$. The cost of the columns and footings will be only very little diminished on account of the reduced number of columns; hence the difference of the above costs per square foot will very closely represent the actual difference.

For light loads probably 16'x16' is the most economical arrangement of bays, while 18'x18' costs only very little more. In the same manner we can compare all other spans and floor loads. The dead load of the entire floor construction per square foot is found by multiplying the figures in the fifth line by 144.

For a given span, 16′x16′ for example, the most economical arrangement of beams is not always the type mentioned at the head of the table. While Type A may be the cheapest for 200 pounds per square foot, Type C is somewhat cheaper for 500 pounds per square foot.

At the head of each span is also given the number of supports required for the form work in one panel, and the story heights for which these supports are safe; these are figured in the average amount of form lumber per square foot.

In applying the average costs, taken from these tables, for the entire floor of a concrete skeleton building, it must be borne in mind that the spandril beams which carry the brick curtain walls are considerably more expensive than the beams in the floor construction; also that the beams at half the girt of the building are omitted, when figuring the average per square foot. The spandril beams are generally of two sizes, viz: 12½″x18″ and 12½″x24″ and reinforced in both cases by about four 1″ round bars, requiring per lineal foot 1.57 cubic feet of concrete, 15 pounds of steel, 11 feet of lumber and 2.1 cubic feet of concrete 15 pounds of steel and 13 feet of lumber, respectively. For checking purposes or for quick estimates, we obtain a fairly close figure for the floor construction of the building if we add to the product of floor area by the average cost of floor per square foot the cost of the spandril beams of a length = 0.7 of the girt of the building. We can do this for every story as well as for the roof, and obtain the cost of the floor construction of the entire building. The cost of the columns can be found very closely by the consideration that the average cost of the columns is very close one cent per lineal foot for each thousand pounds load.

The following example shows how to obtain the cost of columns per square foot of an entire building very quickly:

Let us assume the same building as on page 88.

Floors	Story Heights	Total Height from Basement	Total Floor Loads per Sq. Ft.	Foot Pounds
Roof	12	72	100	7200
5th Floor	12	60	150	6000
4th Floor	12	48	175	8400
3rd Floor	12	36	200	7200
2nd Floor	14	24	250	6000
1st Floor	14	10	300	3000
Basement		Total	1175	40800

In the third column are formed the sums of the story heights from basement up to each particular floor; in the fourth column are noted the total floor loads from the floor above the line on which the figures are printed; in the fifth column are noted the products of the figures in third and fourth columns, and these figures added up give 40800, which represents the number of pounds—lineal feet in the columns per square foot of the building. Every thousand pound-feet costs one cent; hence 40.8c is the cost of the columns per square foot of the building. The cost of footings for a permissible load of 5000 pounds per square foot on the ground is closely 10c for each thousand pounds. By adding the figures in the fourth column, we find as the total of all floor loads per square feet of the building 1175 pounds, or the cost of the footings per square foot of the building 11.75 cents.

In the cost of the columns, determined by this method, is not included the cost due to the weight of the curtain walls. This cost we obtain by substituting in the fourth column for the floor loads the weight of the walls in each story per lineal foot of girt. Then the product of the figures in the third and fourth columns added up gives us the number of pound-feet in the outside columns per lineal foot of girt. In as much as the outside columns are considerably larger than required for strength, it is better to figure the cost of 1000 pound-feet as 1¼c.

Where the outside walls are bearing walls, the cost of the columns and footings, obtained by the above-mentioned method, are to be decreased in the proportion of floor area which is carried by columns to the total floor area of the building.

REINFORCED CONCRETE WALLS.

Reinfored concrete walls should be used in buildings only for pertinent reasons, as, for example, to save floor space, or to take care of wind stresses or to guard against earthquakes. The walls are always more expensive than 12″ brick curtain walls. They are rarely less than four inches, nor more than eight inches thick, and must be reinforced in horizontal direction by ½%, and in vertical direction by ⅛ to ¼%. Openings should preferably have rounded corners, and the walls must be reinforced above the openings by at least 1%, and should have a reinforcement under 45% at all four corners, the same as above the openings.

Reinforced concrete walls are not waterproof, and must be given a coat of R. T. W. paint or cement finish, to prevent the moisture penetrating.

The forms cost, as a rule, more than the concrete and reinforcement taken together.

EXPLANATION OF TABLES ON GIRDERLESS FLOOR CONSTRUCTION.

The great cost of the form-labor and of the additional cost of plastering of beams, caused the designers to reduce the number of beams by using square panel construction, and a further step was the suppression of girders entirely, as shown in Figure 18. The floor in this case is generally supported by columns with flaired capitals, and, in case of end panels, partially by columns and partially by girders or walls. This construction is economical only when the panels are nearly square or when the ratio of the sides of the rectangles, enclosed by the columns, does not exceed 1.33. The problem of calculating the stresses in a plate supported at four points was first thoroughly investigated by the celebrated Prof. Grashof in connection with the strength of endplates in steam boilers, which are held in position by staybolts. He gives the greatest bending moment per lineal foot in such a plate $=\frac{pl^2}{26.5}$, which formula he derived more by an eliminating process than by exact science. By similar reasoning, we lay down the following rule for the computation of girderless floor construction: We divide the panel in strips of a width of 0.35 l, when l is the distance c. c. support in feet; two strips run diagonally, while the others run in the line of the columns. The greatest bending moment per lineal foot of such a strip we assume $=\frac{pl^2}{20}$, when the size of the capital of the supporting column is at least 0.23 l. From this bending moment we easily obtain the required thickness and reinforcing from the slab tables on pages 16 to 20. When the panel is not exactly square we substitute for l the mean of the two sides of the rectangle.

The rods should hang through in the form of a catenary reaching from the underside of the slab in the center to near the top of the slab at the supports. The rods running in the direction of the columns should extend into the adjoining panel 0.12 l, while the diagonal rods should extend 0.16 l beyond the column center.

Comparing the quantities required for this type of floor construction with those given in tables on pages 44-54, we find that the girderless floor constructions take less concrete and steel than where beams and girders are used. It takes also less lumber for the forms, the average for this type being only 2.90 feet of lumber per square foot, and in as much as the unit labor for steel and form work is considerably less than for the other types of construction, the girderless floor is certainly the cheapest of all reinforced floor constructions.

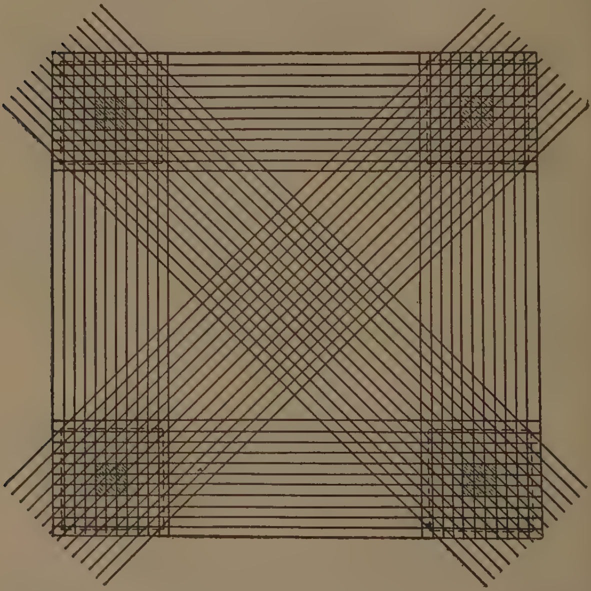


Fig. 18. Showing Strips and Capitals of Columns in Girderless Floor.

DETAILED DESIGN AND COMPUTATION OF COST OF A REINFORCED CONCRETE SKELETON BUILDING.

Let us assume a five-story and basement warehouse building on a lot 67'x200'; and that only two rows of columns are desired in the building. The most economical bay distances for the columns will be 14'x22'. The schedule of page 84 shows how we figure the interior columns. As total load per square foot for each floor we enter the full dead load and the full or a fraction of the live load, according to the requirements of the particular building ordinance. In adding up the floor loads as shown in the fourth column, we can, by means of one setting of the slide-rule, very readily obtain the column loads in the various stories, as these total floor loads must be multiplied with the area of the panel carried by the column=14×22=308 square feet. The size of the column, the reinforcement, the unit quantities of concrete and steel per lineal foot of column, we can directly take from pages 36 to 43, while the form lumber can be taken from page 41. For each story it requires again only one setting of the slide-rule to find the concrete, steel and form lumber for each story, which are entered in the eleventh, twelfth and thirteenth columns. In the fourteenth column is entered the weight of the gas pipe sleeves at each floor level. To the weight of the reinforcement per lineal foot as taken from the tables on pages 36 to 43, must be added the weight of the coil which ties the rods together, which weight=1/20 of the side of columns in inches.

The length of the reinforcing rods for the basement columns must be taken about 18" greater than 10', on account of the extension into the footings.

The totals of the eleventh, twelfth, thirteenth and fourteenth columns give us the quantities in one interior column for the entire height of the building. The outside columns have to carry one-half of the floor loads of the interior columns, which we entered in the third column. A portion of

INTERIOR COLUMNS OF TYPICAL CONCRETE BUILDING.

	Height of Stories Feet	Total Floor Load per sq. ft. Lbs.	Sum of floor loads from roof down to each particular floor. Lbs.	Column Loads in 1000 lbs.	Size of Column Inches	Reinforcement Rounds	Unit Quantities per lineal foot			Quantities for each Story			
							Concrete cubic ft.	Steel lbs.	Form Lumber feet B. M.	Concrete cubic ft.	Steel lbs.	Form Lumber feet B. M.	Gas Pipe Sleeves
Roof		100											
5th Floor	12	150	100	31	$9\frac{5}{8}$x10	4–$\frac{3}{4}$	.67	6.6	8.1	8	79	97	18
4th "	12	175	250	77	$11\frac{5}{8}$x12	4–$\frac{7}{8}$	.96	8.8	8.9	11.6	106	107	27
3rd "	12	200	425	131	$15\frac{1}{4}$x$15\frac{3}{4}$	4–$1\frac{1}{8}$	1.67	14.3	10.7	20.0	172	128	40
2nd "	12	250	625	182	$17\frac{1}{4}$x$17\frac{1}{2}$	4–$1\frac{1}{2}$	2.12	25.0	11.6	25.5	300	139	40
1st "	14	300	875	270	21 x$21\frac{1}{4}$	6–$1\frac{1}{2}$	3.14	37.2	15.0	44.0	520	210	60
Basement	10		1175	362	25 x$25\frac{1}{4}$	8–$1\frac{1}{2}$	4.4	49.3	19.0	44.0	567	190	
Totals for the full height of the Building				.	.	.	.	.	.	153.1	1744	871	185

OUTSIDE COLUMNS.

		$\frac{1}{2}$ Load of Interior Cols. 1000 lbs.	Wall Loads for Bays		Total Loads for Bays										
			14′	22′	14′	22′									
Roof															
5th Floor	12	15.5	7	11	22.5	26.5	23x$12\frac{1}{2}$	4–$\frac{7}{8}$	2.0	9.1	11.6	24	109	142	18
4th "	12	38.5	28	44	66.5	82.5	23x$12\frac{1}{2}$	4–$\frac{7}{8}$	2.0	9.1	11.6	24	109	142	18
3rd "	12	65.5	49	77	114.5	142.5	23x$12\frac{1}{2}$	4–$\frac{7}{8}$	2.0	9.1	11.6	24	109	142	18
2nd "	12	91.0	70	110	161	201	23x$19\frac{1}{2}$	4–$\frac{7}{8}$	3.14	9.1	15.0	37.7	109	142	40
1st "	14	135.0	91	143	244	278	23x$23\frac{1}{4}$	6–$1\frac{1}{2}$	3.70	37.2	18.0	52	520	252	60
Basement	10	181.0	116	182	297	363	25x$25\frac{1}{4}$	8–$1\frac{1}{2}$	4.40	49.3	19.0	44	567	190	
Totals for the full height of the Building						.	.	.	.	.	.	205.7	1523	1010	154

GIRDER DESIGN OF A TYPICAL CONCRETE BUILDING.

Bending Moment in Girders in 1000 Foot Pounds	Beam Number	Width and Depth of Stem of Tee in Inches	Shear at 60 Lbs. per Square Inch	Reinforcement Rounds	Actual Shear at Support Pounds	Difference of Shear Values Pounds	Stirrups Rounds Inches	Stirrups Spacing at Support Inches	Unit Quantities per Lineal Foot Concrete	Unit Quantities per Lineal Foot Steel
$14x325x\frac{1}{10}x20.5^2=192$	62	$11\frac{1}{2}x22\frac{1}{4}$	18800	$6–1\frac{1}{8}$	35000	16200	$\frac{1}{4}$	2″	1.80	27.0
$14x275x\frac{1}{10}x20.5^2=163$	58	$11\frac{1}{2}x20\frac{1}{4}$	17400	$3–1″; 3–1\frac{1}{8}$	29600	12200	$\frac{1}{4}$	3″	1.62	24.0
$14x225x\frac{1}{10}x20.5^2=133$	55	$11\frac{1}{2}x20\frac{1}{4}$	17400	$2-1\frac{1}{8}; 2-1\frac{1}{4}$	24300	6800	$\frac{1}{4}$	5″	1.62	20.0
$14x200x\frac{1}{10}x21^2=124$	49	$11\frac{1}{2}x18\frac{1}{4}$	16000	6–1″	21900	5900	$\frac{1}{4}$	6″	1.45	21.2
$14x160x\frac{1}{10}x21^2=99.5$	42	$11\frac{1}{2}x16\frac{1}{4}$	14600	$4–1\frac{1}{8}$	17500	3900	$\frac{1}{4}$	6″	1.30	17.8
$14x100x\frac{1}{10}x21^2=62.0$	39	$7\frac{1}{2}x16\frac{1}{4}$	9550	$2–\frac{7}{8}–2–1″$	11000	2450	$\frac{1}{4}$	6″	.85	12.4
							Total for all Floors.........		8.64	122.4

SPANDRIL BEAMS 20′ SPANS Bending Moments		1st Floor	2nd Floor	3rd Floor	4th Floor	5th Floor	Roof
$1750x\frac{20^2}{12}=58300$ Foot Pounds	Bending Moments in 1000 Ft. Lbs. From Floor Loads...	96	82	67	62	50	31
	Bending Moments in 1000 Ft. Lbs. From Wall Loads...	58	50	50	50	50	12
$1500x\frac{20^2}{12}=50000$ Foot Pounds	Total Bending Moment..................	154	132	117	112	100	43
	Beam Number (Beam Table)............................	61	57	52	52	52	30
$375x\frac{20^2}{12}=12000$ Foot Pounds	Size of Stem $12\frac{1}{2}$″ by...........................	$22\frac{1}{4}$	$20\frac{1}{4}$	$18\frac{1}{4}$	$18\frac{1}{4}$	$18\frac{1}{4}$	$14\frac{1}{4}$
	Cubic Feet of Concrete per Lineal Foot..................	1.93	1.76	1.60	1.60	1.60	1.24
	Pounds of Steel per Lineal Foot...........................	24	22	22	22	22	10.8

BEAM AND SLAB DESIGN OF A TYPICAL CONCRETE BUILDING.

Bending Moment in Beams in 1000 Foot Pounds	Beam Number	Width and Depth of Stem of Tee in Inches	Shear at 60 Lbs. per Square Inch	Reinforcement Rounds	Actual Shear at Support Pounds	Difference of Shear Values Pounds	Stirrups		Unit Quantities per Lineal Foot			
							Rounds Inches	Spacing at Support Inches	Concrete	Concrete	Steel	Steel
11x325x$\frac{2}{3}$x$\frac{1}{10}$x12.5^2=37.2	15	7½x10¾	6620	4–⅞					.56		10.8	
“ “ “ “ 14^2 47.2	16	“	“	2–⅞;2–1″	11400	4800	¼	4		.56		12.4
11x275x$\frac{2}{3}$x$\frac{1}{10}$x12.5^2 31.6	14	“	“	2–¾;2–⅞					.56		9.5	
“ “ “ “ 14^2 40.0	15	“	“	4–⅞	9600	3000	¼	5		.56		10.8
11x225x$\frac{2}{3}$x$\frac{1}{10}$x12.5^2 25.8	13	“	“	4–¾					.56		7.9	
“ “ “ “ 14^2 32.6	14	“	“	2–¾; 2–⅞	7900	1300	¼	6		.56		9.5
11x200x$\frac{2}{3}$x$\frac{1}{10}$x13^2 24.8	13	“	“	4–¾					.56		7.9	
“ “ “ “ 14^2 29.0	13	“	“	4–¾	7000	400	¼	6		.56		7.9
11x160x$\frac{2}{3}$x$\frac{1}{10}$x13^2 19.9	8	7½x8¾	5700	4–¾					.46		7.9	
“ “ “ “ 14^2 23.3	8	“	“	4–¾	5600		¼	6		.46		7.9
11x100x$\frac{2}{3}$x$\frac{1}{10}$x13^2 12.4	7	“	“	4–⅝					.46		5.4	
“ “ “ “ 14^2 14.5	7	“	“	4–⅝	3500		¼	6		.46		5.4
						Total for all Floors.........			3.16	3.16	42.8	53.9

Bending Moments in Slabs		Thickness		Area of Reinforcement per lin. foot	Rods		Weight of Reinforcement per Sq. Ft. Lbs.
		Inches	Feet		Rounds	Spacing	
Roof	$\frac{11.75^2}{21}$ x 100 = 580	3½	.297	.110	$\frac{5}{16}$	8″	.97
5th Floor	“ 160 = 920	4	.333	.192	$\frac{7}{16}$	9½″	1.60
4th Floor	“ 200 = 1150	4½	.375	.200	“	9″	1.68
3rd Floor	“ 225 = 1300	5	.417	.180	“	10″	1.52
2nd Floor	“ 275 = 1580	5	.417	.240	“	7½″	2.03
1st Floor	“ 325 = 1870	5½	.458	.260	“	7″	2.17
Basement	Total for all Floors............		2.277 cbc. feet				9.97 lbs.

QUANTITIES IN TYPICAL CONCRETE BUILDING.

	UNIT QUANTITIES			TOTAL QUANTITIES		
	Concrete Cb. Ft.	Steel Lbs.	Form Lumber Ft. B. M.	Concrete Cubic Feet	Steel Pounds	Form Lumber Feet Board Measure
26 Interior columns	153.1	1929	871	3980	50100	22600
34 Footings, 8′-6″ square	110.0	230	34	3740	7820	1160
26 Outside columns, 14′ bays	205.7	1677	1010	5340	43500	26300
8 Outside columns, 22′ bays	198.7	1537	1000	1590	12300	8000
26 Footings, 8′ square	90	200	32	2340	5200	840
13 Rows of girders 65′ lg,=845′	8.64	122.4		7300	104000	
2 Rows of beams 200′ lg,=400′	3.16	42.8		1264	17200	
3 Rows of Beams 200′ lg,=600′	3.16	53.9		1900	32300	
67x200=13400 square feet of floor area	2.277	9.97	19.92*	30600	134000	268000
65′ Of spandril beams 1st floor	1.93	24.	13	126	1560	850
130′ Of spandril beams 2nd floor	1.76	22.	13	230	2860	1700
390′ Of spandril beams 3rd, 4th and 5th floor	1.60	22.	11	624	8600	4300
130′ Of spandril beams, roof	1.24	10.8	10	160	1400	1300
400′ Of spandril beams, 2nd floor 14′ Span	1.76	10.8	13	700	4320	5200
1200′ Of spandril beams, 3rd, 4th and 5th floor	1.60	10.8	11	1920	13000	13200
400′ Of spandril beams, roof	1.24	10.8	10	500	4320	4000
430 Lineal feet of basement walls (on three sides of the building)	6.0	20.0	44	2580	8600	18900
Total for the structural work of the entire building				64894	451080	376350

64894 Cubic feet of concrete @ 25 cents	$16,223.50
451080 Pounds of steel @ 2½ cents	11,271.00
376350 Feet of lumber @ 4.0 cents	15,054.00
Total cost of structural work	$42,548.50

The cubical contents of the building from basement to roof=13400×72=946,800 cubic feet.

* This item is taken from the tables of typical floor construction on page 46.

the outside columns have to carry the wall loads for a length of 14′, while others have to carry wall loads for a length of 22′. These wall loads for each story were figured and the total loads up to each particular story were entered into the next two columns. These loads we have to add to the floor loads given in the third column, and the results are entered in the next two columns. The sizes of the columns and the other data can again be found in the pages 36 to 43, but it must be considered that it is desirable to have the width of the columns above the first floor uniform, and that the depth of the columns cannot be less than 12½″, which is the thickness of the brick walls. We shall make both kinds of columns alike above the second floor, but below the second floor we shall adopt for basement and first floor, for the smaller bay, a column 23″x23¼″, reinforced by six 1½″ and six 1″ round bars, respectively, which will decrease the total quantities given for the larger bay to 198.7 cubic feet of concrete, 1537 pounds of steel including sleeves, and 1000 feet of lumber.

From page 34 we find that the footings for all interior columns and the outside columns of 22′ bays must be 8′ 6″ square for a permissible load of 5000 pounds per square foot on the ground, and that each footing contains 110 cubic feet of concrete and 230 pounds of steel; while the outside columns of the 14′ bays may have 8′ footings, containing 90 cubic feet of concrete and 200 pounds of steel.

In figuring the floor construction we shall assume somewhat higher floor loads than for the columns. On page 85 is shown the schedule for the girder and beam calculation. We shall adopt Type A with slabs, supported on four sides. As span of the girders we assume 20.5′ and 21′, respectively, while for the beams between columns we assume 12.5 and 13′, and for the beams, carried by girders, we assume the span=14′.

After having found the bending moments, all the data in the next five columns can be found in the beam table on pages 6 to 8. To facilitate the form work, all girders from first to fifth floor are of the same width, and on account of fireproofing no beam was made of a width less than 7½″.

In figuring the shear for the girders it is to be considered that the load carried by the girders is only ¾ of the entire panel load on account of the square slabs, and in figuring the shear of the beams it must be considered that each slab transmits only ¼ of the load to each beam. After having found the shear and substracted from it the shear given in the beam tables, the spacing of the stirrups is found by the help of the table on page 62.

The spandril beams of 22′ span have to support ½ of the floor loads of the girders, and in figuring the bending moment from the wall loads we can use the formula $\frac{pl^2}{12}$. For the first story the wall load=1750 pounds per lineal foot; for the second to fourth story, 1500 pounds; and for the roof, only 375 pounds per foot from the parapet wall. These bending moments were added on page 85 to the bending moments from the floor loads, and in determining the beams we have to make some allowance for the fact that the flange of the Tee exists only on one side of the spandril beams, and therefore somewhat larger beams should be adopted than corresponding to the bending moments in the schedule.

The lintels in the 14′ bays have very little to carry, and we shall make them the same size as those in the 22′ bays, but reinforced only by four ⅞″ round bars. The slabs are figured on page 86.

On page 114 we find that for the reinforced concrete basement walls we can adopt a 6″ wall reinforced by 0.28 square inches per lineal foot in vertical direction, and by 0.18 square inches per lineal foot as longitudinal reinforcement, or a total reinforcement of 1.8 pounds per square foot. Including footing, one foot wide, we shall assume the quantities per lineal foot of retaining wall with 6 cubic feet on concrete, 20 pounds of steel, and 44 feet of form lumber.

On page 87 is shown the schedule of quantities for the entire building, and figuring the cost of the concrete at 25c per cubic foot, the cost of the steel at 2½c per pound, and the cost of the forms at $40 per M feet B. M., we obtain the cost of the rough structural work in the building at $42,548.50.

To this is to be added the cost of the brick walls, which in this case will be in the neighborhood of $8,500, and the cost of the basement floor, which does not vary much from 12c per square foot. If the floors are to be cement finished, we have to substitute for a thickness of ½" rough concrete a finishing coat, which generally takes more than the average of ½"; it will be near enough to add to the figures on page 87 4c for each square foot of finished floor. If a wooden floor is specified, sleepers are to be imbedded on top of the rough floor and held down by a cinder concrete filling, generally 1½" thick. The cinder concrete is generally mixed in the proportion 1:9 and averages 1½c per square foot.

Adding the cost of the brick walls ($8,500), of the basement floor ($1,570), of the cement finish on all floors ($2,600), of the stairs (about $750), and $6000 for contractors' profit, machinery, tools, etc., we obtain as grand total for the rough building in round figures $62,000, or 6.5 cts per cubic foot of contents.

EXPLANATION OF TABLES ON FORM WORK.

Next to the proper design of the structural part the proper design of the form work is the most important part of reinforced concrete construction. The cost of the forms amounts in an average to 33% of the cost of the entire work under good management, and often reaches 60% under inexperienced management. It is a fact that it takes more lumber to build a reinforced concrete building than an ordinary mill construction building and that the unit labor per 1000 feet., B. M., is more than twice as high, yet, it is surprising to see in how few instances the form work is designed by competent persons, and practically any foreman, who declares himself competent, is given charge of the design of fully 33% of the work in the building. This, in connection with the fact that almost any contractor, who has the courage to bid on a reinforced concrete structure, is awarded the contract for its execution, as long as he is the low bidder, should cause the designing engineer or architect to design also the form work.

The forms must be designed not only for strength but also for stiffness. Girders and beams look very unsightly if they have a sag or show bulging of the forms, or the loss of concrete by the deflection of the form work in floors and walls may be quite considerable.

The forms are only temporary structures; therefore, we may allow a stress of 1800 pounds per square inch in bending and may use a factor of safety of three for the supports.

The tables on pages 92-94 give the safe loads on boards and planks of various thickness and 12″ wide, of joists of various depths, while on page 95 will be found the safe load on supports.

Applying these tables for the design of the forms for floor slabs, we can readily see that ⅞″ flooring is amply strong for all ordinary floor loads, as a ⅞″ board, 12″ wide, will support, on a span of 2 feet, a load of 1375 pounds with a deflection of about 1/20″. This load can only be produced by the large

SAFE LOADS AND DEFLECTIONS OF JOISTS

Joists $1\frac{5}{8}$ Thick Depth in Inches	SPAN						
	1′-6″	1′-9″	2′	2′-3″	2′-6″	2′-9″	3′
$3\frac{1}{2}$	**2670** .0225 .009	**2280** .031 .0124	**2000** .04 .016	**1780** .051 .020	**1600** .063 .025	**1450** .076 .030	**1330** .09 .036
	3′-6″	4′	4′-6″	5′	5′-6″	6′	6′-6″
$5\frac{1}{2}$	**2800** .080 .032	**2450** .104 .042	**2170** .132 .053	**1960** .163 .065	**1780** .196 .080	**1630** .234 .094	**1500** .273 .110
$7\frac{1}{2}$	**5200** .059 .023	**4550** .077 .031	**4070** .097 .039	**3660** .120 .048	**3330** .146 .058	**3050** .174 .070	**2800** .202 .081
$9\frac{1}{2}$	**8300** .046 .018	**7300** .061 .025	**6480** .077 .031	**5800** .095 .038	**5300** .115 .046	**4850** .137 .055	**4480** .161 .065
	6′	6′-6″	7′	7′-6″	8′	8′-6″	9′
$11\frac{1}{2}$	**7200** .113 .046	**6630** .132 .053	**6140** .154 .062	**5730** .176 .071	**5390** .202 .081	5050 .226 .091	**4790** .255 .102
$13\frac{1}{2}$	**9800** .096 .039	**9100** .112 .045	**8450** .130 .052	**7850** .149 .060	**7350** .170 .068	**6950** .191 .077	**6550** .215 .086

The black figures denote the safe load at 1800 pounds stress.
supported. The figures in the third line are deflections for

For ⅞″ joists multiply above safe loads by 0.54.

For the full sections....................................
Multiply the loads of the corresponding short sizes by....
Multiply the deflections by ....

For continuously supported joists multiply the loads by 1.5

SUPPORTED BY TWO OR THREE SUPPORTS.

IN FEET								
3′-6″	**4′**	**4′-6″**	**5′**	**5′-6″**	**6′**	**6′-6″**	**7′**	**8′**
1140 .1225 .049	**1000** .160 .064	**890** .20 .08	**800** .25 .10	**725** .30 .12	**670** .36 .144	**615** .42 .168	**570** .49 .196	**500** .64 .254
7′	**7′-6″**	**8′**	**8′-6″**	**9′**	**10′**	**11′**	**12′**	**13′**
1400 .320 .130	**1300** .365 .146	**1220** .415 .166	**1150** .470 .190	**1080** .525 .210	**980** .650 .260	**900** .790 .320		
2620 .235 .094	**2430** .270 .108	**2260** .310 .124	**2150** .345 .138	**2030** .390 .156	**1830** .480 .192	**1660** .580 .232	**1520** .690 .276	**1400** .810 .324
4160 .187 .075	**3880** .215 .086	**3630** .244 .098	**3430** .275 .110	**3230** .310 .124	**2920** .380 .152	**2650** .460 .184	**2420** .550 .220	**2250** .645 .260
9′-6″	**10′**	**10′-6′**	**11′**	**11′-6″**	**12′**	**13′**	**14′**	**15′**
4540 .283 .114	**4300** .313 .126	**4070** .346 .137	**3920** .380 .152	**3730** .415 .167	**3580** .450 .181	**3320** .530 .213	**3070** .615 .246	**2870** .705 .282
C200 .240 .096	**5900** .267 .107	**5620** .295 .118	**5360** .323 .129	**5140** .353 .142	**4920** .383 .154	**4550** .450 .180	**4200** .520 .209	**3950** .600 .240

The figures below the black denote the deflections, if freely three supports.

For 1⅜″ joists multiply above safe loads by 0.85.

2″x4″	2″x6″	2″x8″	2″x10″	2″x12″	2″x14″
1.61	1.46	1.40	1.37	1.34	1.32
0.87	0.92	0.94	0.95	0.96	0.97

and the deflections by 0.75.

SAFE LOADS AND DEFLECTIONS OF BOARDS AND PLANKS 12″ WIDE.

Thickness of Boards		SPAN IN FEET															
		1′-0″	1′-3″	1′-6″	1′-9″	2′-0″	2′-3″	2′-6″	2′-9″	3′-0″	′3-6″	4′-0′	4′-6″	5′-0″	6′-0″	7′-0″	8′-0″
$\frac{7}{8}$	Continuous	2750 .0124	2200 .0194	1840 .028	1570 .038	1375 .050	1220 .063	1100 .077	1000 .094	920 .112	780 .153	690 .20	610 .25	550 .31	456 .45	393 .61	345 .80
	Non-continuous	1830 .0414	1460 .0647	1230 .0933	1040 .127	915 .167	813 .210	733 .256	667 .313	611 .366	519 .510	459 .665	406 .833	366 1.04	304 1.50	262 2.03	230 2.67
$1\frac{3}{8}$	Continuous	6750 .0079	5400 .0123	4550 .0177	3850 .0236	3370 .0318	3020 .0401	2700 .0495	2450 .059	2270 .071	1920 .097	1690 .126	1490 .160	1350 .197	1120 .283	968 .390	845 .508
	Non-continuous	4500 .0264	3600 .041	3030 .059	2560 .079	2250 .106	2010 .134	1800 .165	1630 .197	1520 .236	1280 .322	1120 .420	990 .530	900 .660	748 .95	642 1.30	561 1.69
$1\frac{5}{8}$	Continuous	9500 .0067	7600 .0104	6400 .015	5420 .020	4750 .027	4250 .034	3800 .042	3450 .050	3200 .060	2700 .082	2380 .107	2100 .135	1900 .167	1580 .240	1360 .330	1190 .43
	Non-continuous	6320 .0223	5080 .0346	4270 .050	3620 .0667	3170 .090	2830 .114	2530 .140	2300 .167	2140 .200	1800 .273	1580 .357	1400 .450	1270 .560	1050 .80	910 1.10	790 1.44
2″	Continuous	14300 .0054	11400 .0084	9650 .0122	8180 .0162	7180 .022	6410 .0275	5730 .034	5200 .041	4830 .049	4080 .067	3600 .087	3170 .11	2870 .136	2380 .195	2050 .267	1800 .350
	Non-continuous	9500 .0181	7600 .0281	6400 .0406	5420 .0541	4750 .0730	4250 .0920	3800 .114	3450 .135	3200 .162	2700 .221	2380 .280	2100 .365	1900 .452	1580 .650	1360 .892	1190 1.16

Fibre stress is assumed = 1800 pounds per square inch. ‘‘Continuous’’ means boards supported on four or more supports; bending moment assumed $\frac{pl^2}{12}$ ‘‘Non-continuous’’ means boards supported on two or three supports; bending moment assumed $\frac{pl^2}{8}$

The deflections given under heading ‘‘Non-continuous’’ are for boards supported by three supports. For freely supported boards the deflection are 2½ greater than the latter.

SAFE LOADS AND DEFLECTIONS FOR GIRDERS CARRYING JOISTS.

Width and Depth of Girder	SPAN IN FEET								
	5′	5′–6″	6′	6′–6″	7′	7′-6″	8′	9′	10′
2⅝″ X 5⅝″	3400 .064	3100 .078	2800 .092	2600 .108	2400 .1250	The deflections are for semi-continuous girders.			
3″ x 6″	4300 .060	3800 .072	3500 .086	3250 .100	3000 .117				
2⅝″ X 7⅝″	6200 .047	5600 .057	5200 .067	4800 .079	4400 .092	4100 .105	3900 .120	3450 .152	3100 .188
3″ x 8″	7600 .045	7000 .054	6400 .065	5900 .075	5400 .088	5100 .101	4800 .115	4300 .146	3900 .180

SAFE LOADS ON SUPPORTS

Unsupported length in feet to least width of support in Inches	Safe load per sq. inch of cross section pounds
2.0	720
2.1	654
2.2	600
2.3	548
2.4	500
2.5	460
2.6	427
2.7	398
2.8	367
2.9	345
3.0	323
3.1	300
3.2	282
3.3	265
3.4	250
3.5	237
3.8	200
4.0	180
4.2	164
4.4	148
4.6	137
4.8	125
5.0	117

Supports	Unsupported Length					
	9′	10′	11′	12′	13′	14′
3⅝″x3⅝″ 4″x4″	6.1 9.2	5.0 7.4	4.1 6.1	3.3 5.2	3.0 4.4	2.60 3.8
	16′	17′	18′	19′	20′	21′
5⅝″x5⅝″ 6″x6″	10.7 15.0	9.6 13.0	8.4 11.6	7.6 10.2	6.9 9.3	6.3 8.5
	22′	23′	24′	26′	27′	28′
5⅝″x5⅝″ 6″x6″	5.7 7.8	5.4 8.1	4.8 6.50	3.7 5.5	3.8 5.1	3.5 4.8

The black figures denote the permissible load in 1000 pounds. Where a strut is braced at mid height, take as unsupported length ¾ of the height. Where a strut is braced at the two points ⅓ of the height apart take as unsupported length ½ of the total height.

DATA ON BEAM FORMS.

Size of Beam	Nominal Sizes of Lumber Used		Feet Board Measure per Lineal Foot
	in Bottom SIS&IE	in One Side SIS	
3½x 6¾	2 x 4	2x8	4.00
4 x 7½	1½x 4	1½x8	3.33
5½x 8¾	2 x 6	2x10	5.00
6 x 9½	1½x 6	1½x10	3.92
7½x 8¾	2 x 8	2x10	5.34
8 x 9½	1½x 8	1½x10	4.17
5½x10¾	2 x 6	2x12	5.67
6 x11½	1½x 6	1½x12	4.42
7½x10¾	2 x 8	2x12	6.00
8 x11½	1½x 8	1½x12	4.67
9½x10¾	2 x10	2x12	6.34
10 x11½	1½x10	1½x12	4.92
7½x12¾	2 x 8	2x14	6.67
8 x13½	1½x 8	1½x14	5.17
9½x12¾	2 x10	2x14	7.00
10 x13½	1½x10	1½x14	5.42
7½x14¼	2 x 8	2–2x8	7.67
8 x15½	1½x 8	2–1½x8	6.00
9½x14¼	2 x10	2–2x8	8.00
10 x15½	1½x10	2–1½x8	6.25
11½x14¼	2 x12	2–2x8	8.33
12 x15½	2 x12	2–1½x8	7.00
7½x16¼	2 x 8	1–2x8,1–2x10	8.33
8 x17½	1½x 8	1–1½x8,1–1½x10	6.50
9½x16¼	2 x10	1–2x8,1–2x10	8.67
10 x17½	1½x10	1–1½x8,1–1½x10	6.75

Where the length of the forms is not in even feet, we shall have a certain waste in cutting the commercial lengths of lumber, which waste may reach from 10 to 20% of the values given in the table. A girder or beam form has rarely to carry more than 600 to 800 pounds per lineal foot, and by

DATA ON BEAM FORMS.

Size of Beam	Nominal Sizes of Lumber Used		Feet Board Measure per Lineal Foot	Stiffeners
	In Bottom S1S&1E	in One Side S1S		
11½x16¼	2 x12	1–2x8,1–2x10	9.00	
12 x17½	2 x12	1–1½x8,1–1½x10	7.50	
9½x18¼	2 x10	2–2x10	9.33	
10 x19½	1½x10	2–1½x10	7.25	
11½x18¼	2 x12	2–2x10	9.67	
12 x19½	2 x12	2–1½x10	8.00	
9½x20¼	2 x10	1–2x10,1–2x12	10.00	
10 x21½	1½x10	1–1½x10,1–1½x12	7.75	
11½x20¼	2 x12	1–2x10,1–2x12	10.33	
12 x21½	2 x12	1–1½x10,1–1½x12	8.50	
11½x22¼	2 x12	2–2x12	11.00	
12 x23½	2 x12	2–1½x12	9.00	
11½x26	2 x12	2–1x10,1–1x8	8.72	2″x4″–3′lg 4′0″c.c.
13½x26	2 x14	2–1x10,1–1x8	8.92	
11½x28	2 x12	3–1x10	8.92	"
13½x28	2 x14	3–1x10	9.25	"
11½x30	2 x12	4–1x8	9.73	2″x4″–3′–9″lg
13½x30	2 x14	4–1x8	10.07	3′–6″c.c.
13½x36	2 x14	4–1x10	11.63	2″x4″–4′lg
13½x42	2 x14	4–1x10,1–1x6	12.85	3′–3″c.c.
15 x42	2–2x8	4–1x10,1–1x6	13.35	2″x4″–4′–6″lg
15 x48	2–2x8	4–1x10,2–1x6	14.53	2″x4″–5′lg
17 x48	1-2x10 1–2x8	4–1x10,2–1x6	14.87	3′–0″c.c.
15 x52	2–2x8	4–1x10,2–1x8	15.45	2″x4″–5′–6″lg
17 x52	1–2x8 1-2x10	4–1x10,2–1x8	15.78	3′–0″c.c.

inspecting the tables on pages 92 and 93, it will be found that the side boards can carry this load on six feet and more with deflections less than 1/10″; therefore, it is not necessary to place supports nearer than 5′ on centers under the girder forms.

Fig. 19

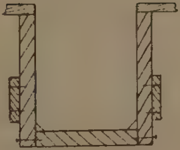


Fig. 20

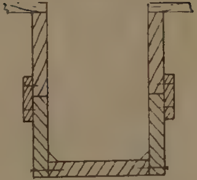


Fig. 21

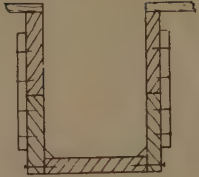


Fig. 22

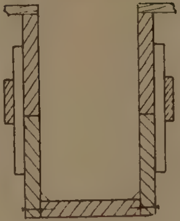
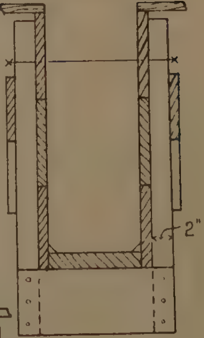


Fig. 23



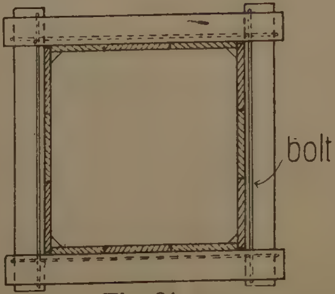


Fig. 24

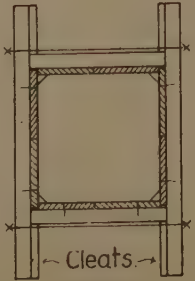


Fig. 25

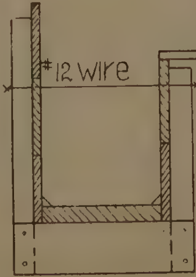


Fig. 26

Girder and Column Forms.

ARRANGEMENT OF FORMS FOR TYPICAL FLOOR CONSTRUCTIONS.

Spacing of Bays	Joists Width and Depth	Girders Width and Depth	Number of 4″x4″ Sup't's	Figure Number
12′x12′	2″x6″	3″x6″	4	1
12x14	2x6†	3x6	4	1
12x16	2x6†	3x8	4	1
12x18	2x8		4	3
12x20	2x4†	3x6	7	4
12x24	2x6	3x6	7	4
14x14	2x6†	3x6	4	1
14x16	2x6†	3x8	4†	1
14x18	2x6†		6	9
14x20	2x6		6†	9
14x22	2x6	3x6	7†	4
14x24	2x6†	3x6	7†	4
14x26	2x6	3x8	9†	5
14x28	2x6	3x8	9†	5
14x30	2x6	3x8	10†*	6
14x35	2x6	3x8	11†*	11
14x40	2x6	3x8	11†*	11
14x50	2x6	3x8	16†*	14
16x16	2x6†	3x8†	4†	1
16x18	2x8		6	9
16x20	2x8		6†	9
16x22	2x8		6†	9
16x24	2x8		8†	10
16x26	2x8		8†	10
16x28	2x6	3x8†	11	7
16x30	2x6	3x8†	11†*	7
16x35	2x6	3x8†	14†*	12
16x40	2x6	3x8†	14†*	12
16x50	2x6	3x8†	19†*	17
18′x18′	2″x6″	4″x8″†	5†	2
18x20	2x6		10†	18
18x22	2x6		10†	18
18x24	2x6		10†	18
18x26	2x6	3x8	12†	8
18x28	2x6	3x8	12†	8
18x30	2x4	3x6†	19†*	13
18x35	2x6	3x6	19†*	13
18x40	2x6	3x6†	19†*	13
18x50	2x6	3x6†	26†*	18
20x20	2x6		10†*	18
20x22	2x6		10†*	18
20x24	2x6		10†*	18
20x26	2x6†		10†*	19
30x28	2x8		13†*	19
20x30	2x8†		13†*	17
20x40	2x8		19†*	21
20x50	2x8		23†*	18
22x22	2x6†		10†*	18
22x24	2x6†		10†*	19
22x26	2x8		13†*	19
22x28	2x8†		13†*	20
22x30	2x8		16†*	19
24x24	2x6†		13†*	20
24x26	2x6†		16†*	20
24x28	2x8†		16†*	20
24x30	2x8†		16†*	

† Denotes that full size of lumber must be used.

* Denotes that the 4″x4″ must be braced at mid-height by 1″x6″ in both directions.

The figures numbers refer to pages 100 and 101.

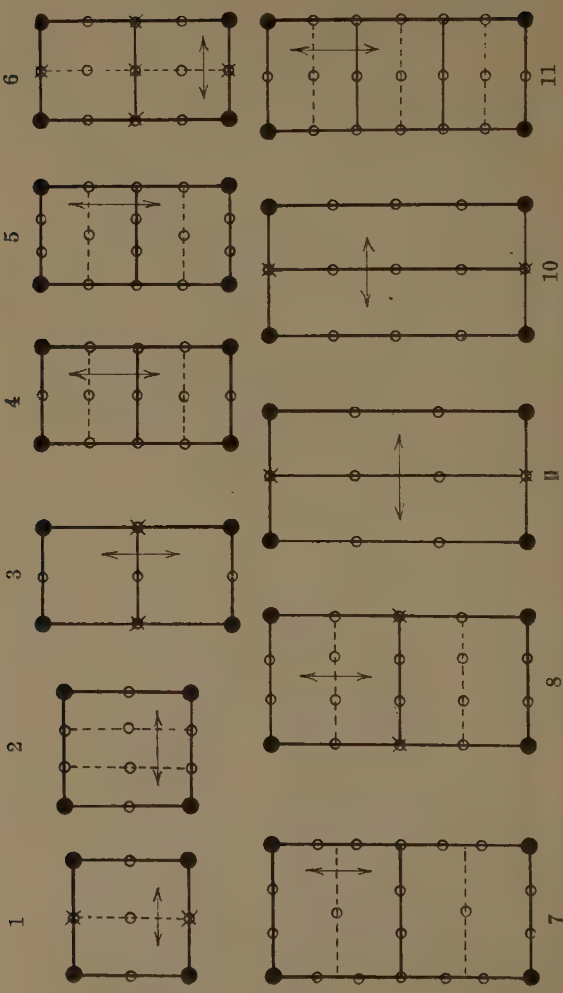


Fig. 35 to 45. Showing Distribution of Supports in Typical Floor Constructions.

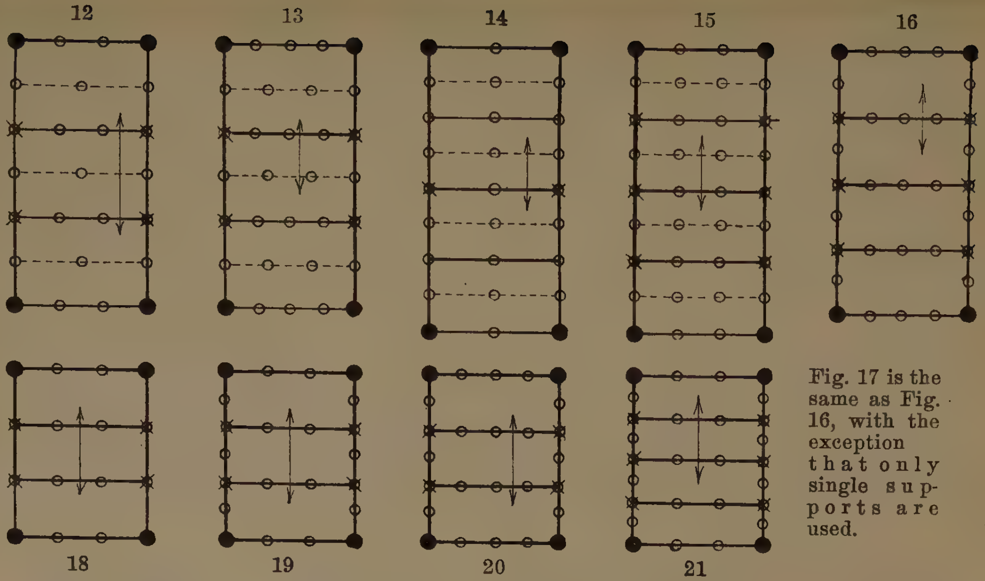


Fig. 17 is the same as Fig. 16, with the exception that only single supports are used.

Fig. 46 to Fig. 54. Showing Distribution of Supports in Typical Floor Construction.

DATA ON COLUMN FORMS.

Size of Colum in Inches	Boards Used in Shorter Sides SIS & 2E	Boards Used in Longer Sides SIS & 2E	FRAMES			Form Lumber per Lineal Foot Including Braces	Stress on One Side of Frame	Size of Bolts
			Number	Size Inches	Length			
9⅝x10″	2–⅞″x9⅝″	4–⅞″x5⅝″	2 2	2x4″ 2x4	2′–6″ 0′–11¼″	8.1	1200	7/16″
11½x12	2–⅞x11½	2–⅞x5⅝ 2–⅞x7⅝	2 2	2x4 2x4	2–8 1–1	8.9	1400	7/16
13¼x13¾	2–⅞x7⅝ 2–⅞x5⅝	4–⅞x7⅝	2 2	2x4 2x4	2–10 1–2⅞	9.7	1600	½
15¼x15¾	4–⅞x7⅝	2–⅞x9⅝ 2–⅞x7⅝	2 2	2x4 2x4	3–0 1–4⅞	10.7	1840	⅝
17¼x17½	2–⅞x7⅝ 2–⅞x9⅝	4–⅞x9⅝	2 2	2x4 2x4	3–2 1–6⅞	11.6	2050	⅝
19¼x19½	4–⅞x9⅝	2–⅞x5⅝ 4–⅞x7⅝	2 2	2x6 2x6	3–4 1–8⅞	14.0	2280	⅝
21x21½	4–⅞x7⅝ 2–⅞x5⅝	6–⅞x7⅝	2 2	2x6 2x6	3–6 1–10⅞	15.	2500	⅝
23x23¼	6–⅞x7⅝	4–⅞x7⅝ 2–⅞x9⅝	2 2	2x6 2x6	3–8 2–0⅞	18.	2050	⅝
25x25¼	4–⅞x7⅝ 2–⅞x9⅝	2–⅞x7⅝ 4–⅞x9⅝	2 2	2x6 2x6	3–10 2–2⅞	19.	2200	⅝
27x27¼	2–⅞x7⅝ 4–⅞x9⅝	6–⅞x9⅝	4	4x4	3–3	20.4	3200	⅝
29x29¼	6–⅞x9⅝	8–⅞x7⅝	4	4x4	3–6	21.8	3400	⅝
31x31	8–⅞x7⅝	6–⅞x7⅝ 2–⅞x9⅝	4	4x4	3–8	23.	3600	⅝
33x33	6–⅞x7⅝ 2–⅞x9⅝	4–⅞x7⅝ 4–⅞x9⅝	4	4x4	3–10	24.3	3850	⅝
13.5 Octag.	8–1⅜x5⅝		Similar to 13¼x13¾			10.8		½
18.5 Octag.	8–1⅜x7⅝		Similar to 19¼x19½			15.1		⅝
23.3 Octag.	8–1⅜x9⅝		4	4x4	3′–0″	20.0		⅝
28.0 Octag.	8–1⅜x11½		4	4x4	3–6	23.6		⅝

The frames are two feet c. c., except in 23x23¼ and 25x25¼, where they are 18″ c. c.

All 2″x4″, 2″x6″ and 4″x4″ are rough.

FREE SPANS IN BOARDS FOR COLUMN AND WALL FORMS.

Depth of Wet Concrete in Feet	Pressure in Pounds per Square Foot	Greatest Free Span of Column Boards			Greatest Free Span of Wall Boards		
		Thickness			Thickness		
		$\frac{7}{8}''$	$1\frac{3}{8}''$	$1\frac{5}{8}''$	$\frac{7}{8}''$	$1\frac{3}{8}''$	$1\frac{5}{8}''$
1	70				4′–0″	5′–6″	6′–0″
2	140	2′–6″			3–6	5–0	5–9
3	210	2–6	3′–6″		3–3	4–7	5–3
4	280	2–3	3–4	3′–9″	3–1	4–5	4–9
5	350	2–0	3–0	3–6	2–9	4–4	4–6
6	420	2–0	2–10	3–4	2–6	4–0	4–5
7	490	1–9	2–9	3–3	2–4	3–8	4–1
8	560	1–9	2–8	3–0	2–2	3–4	3–10
9	630	1–6	2–7	2–11	2–0	3–3	3–8
10	700	1–6	2–7	2–10	1–11	3–1	3–6
11	770	1–6	2–7	2–10	1–10	2–11	3–4
12	840	1–6	2–6	2–9	1–9	2–10	3–2
13	910	1–6	2–6	2–9	1–8	2–9	3–1
14	980	1–6	2–6	2–8	1–8	2–7	3–0
15	1050	1–6	2–6	2–7	1–7	2–6	2–10
16	1120	1–6	2–5	2–6	1–7	2–5	2–9
17	1190	1–6	2–5	2–6	1–6	2–4	2–9
18	1260	1–5	2–4	2–5	1–5	2–3	2–8
19	1330	1–5	2–3	2–4	1–5	2–3	2–8
20	1400	1–4	2–2	2–4	1–4	2–2	2–7

We assume that the boards are not stressed more than 1800 pounds per square inch, and that the deflection of the boards are limited in columns to 1/30″, and in walls to ⅛″.

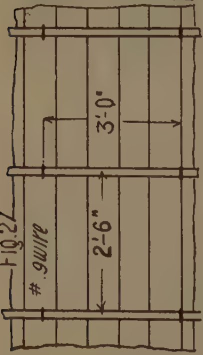


Fig. 27

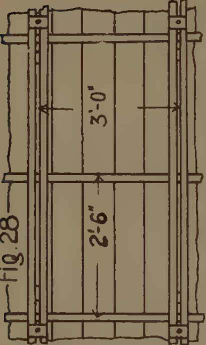


Fig. 28

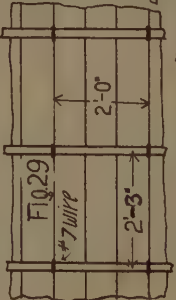


Fig. 29

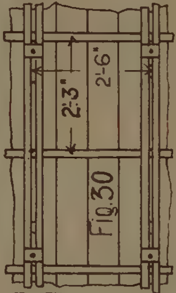
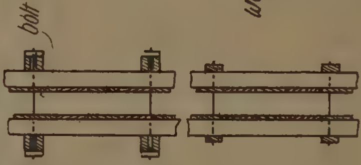


Fig. 30



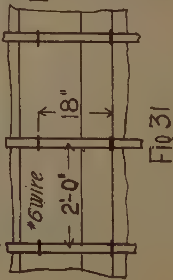


Fig. 31

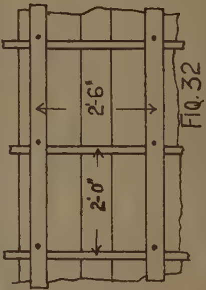


Fig. 32

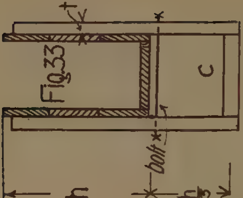


Fig. 33

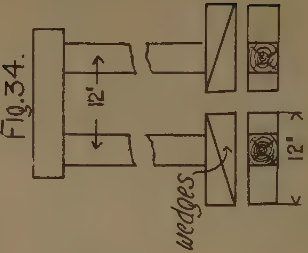


Fig. 34.

Wall Forms and Special Lintel Form.

carts which are at present used for wheeling of the concrete, and experience proves that very little breakage occurs in the ⅞″ flooring.

The floor boards must be supported by joists. These joists should be surfaced on one edge to make them all of the same depth, and should be laid with the crown up. The joists are either carried entirely by ledger boards, nailed to the side of the beam forms, or partially by ledger boards and partially by 3″x6″ or 3″x8″ girders. The deflections in joists should be limited to ¼″, and assuming that each joists has to carry per lineal foot a load of 160 pounds, which corresponds to the weight of a 5″ slab and a live load of 20 pounds per square foot, where the joists are spaced 2′ c.c., or to the weight of a 6″ slab and 20 pounds live load, where the joists are spaced 19″ c. c., we can find the clear spans which do not stress the lumber to more than 1800 pounds per square inch, or deflect the joists more than ¼″ in the following table:

Size of Joists, Inches.....	1⅝x3½	2x3⅞	1⅝x5½	2x5⅞	1⅝x7⅝	2x7⅞	1⅝x9⅝	2x9⅞
Span, if freely supported	5′	5′–7″	6′–11″	8′–0″	8′–8″	9′–6″	9′–11″	10′–8″
Span, sup'td on 3 sup'rts.	5′	6′–3″	7′–8″	9′–6″	10′–7″	11′–9″	12′–9″	13′–2″

When the spans exceed the figures given for freely supported joists, they must be supported by 3″x6″, or 3″x8″ girders. These girders always are supported in the center by a strut in order to limit their span to 6′ to 9′, and the exact spans can be taken from table on page 92.

The tables on pages 96, 97, and Figures 19-23, on page 98, give the data for the beam forms for all beams given in beam tables on pages 6 to 10.

The depths 6¾″, 8¾″, 10¾″ 12¾″, 14¼″, etc., refer to beams No. 1 to No. 65 of the beam tables, and are to be used where the commercial sizes of lumber are less than the nominal sizes, while the depths of 7½, 9½, 11½, 13½, 15½, etc., refer to beams No. 201 to No. 265 and are to be used where full sized lumber is obtainable.

Figure 19 shows the form for a beam 10¾″ deep. This depth is obtained as follows: We add to width of the nominal 12″ plank, which is actually only 11⅝″, the thickness of the floor boards=⅞″, and deduct the thickness of the bottom plank, which in most cases is 1¾″, and obtain 10¾″, depth of the stem of the concrete beam. As a rule it is unnecessary to use as thick planks as 1¾ or 1⅝″; planks 1⅜″ thick are amply strong in most cases. They are, however, rarely kept in stock, and can as a rule only be obtained by resawing of 3″ planks. Because they are more rarely used (although their use represents a considerable saving), we assume their use only in case of full sizes of lumber. Here the depth of 11½″ is obtained by adding to 12″ the thickness of the floor boards=⅞″ and deducting 1⅜″, the thickness of the bottom boards.

In beams of a greater depth of 12¾″ or 13½″, two planks must be used for the sides of the beams. For depths up to 16¼″ or 17½″ these two planks may be held together by the 1″x4″ (rough) ledger boards, as shown in Figure 20, which ledger boards are nailed to both planks by 6d. nails, 6″ c.c. The side boards are nailed to the bottom boards by 16d. or 12d. nails (according as to whether 1⅝″ or 1⅜″ planks are used), 18″ c.c. These nails should not be driven home, but the heads should be left projecting about ⅛″, to facilitate the taking down of the forms. Where no ledger boards are used the sides must be held together by 1″x3″ or 1″x4″ cleats, spaced about 4′ 0″ c.c., as shown in Figure 21. Figure 22 shows beam forms up to 21½″ depths. The sides must be held together by 1″x4″ cleats, 2′ 0″ c.c., which cleats are about 12″ long and nailed to the side boards by eight 6d. nails, and to these cleats the 1″x4″ ledger boards are nailed by six 6d nails.

Above 21½″ we recommend the use of ⅞″ boards for the sides, to which sides are nailed at distances given in table, rough 2″x4″ cleats, which are tied together at the bottom of the beam by two pieces 1″x6″ or 1″x8″, as shown in Figure 23. It is well to wire the 2″x4″ cleats after the steel bars are placed, in order to prevent the spreading of forms by the pressure of the concrete. The ledger boards should be 1″x6″ and nailed to the 2″x4″ cleats, and should be further supported by 1″x6″ bracket pieces, about 8″ long.

For spandril beams and sometimes for deep girders, a design as shown in Figure 33 is of advantage. The cleats are nailed to the sides and hold the sides together by bolts and distances pieces, "c." The cleats must be figured for deflection by the formula. Deflection in inches$=\frac{n}{75}\left(\frac{h}{10}\right)^5 x \frac{1}{wt^3}$, when n the distance of the cleats c.c. in feet, h the depth of wet concrete in inches, w the width parallel to the beam in inches and t the thickness of the cleat under right angle to the beam. For 1⅝x3⅝ flat and on edge and for 2″x4″ rough, flat and on edge, the formula reduces to $\frac{n}{1960}\left(\frac{h}{10}\right)^5$, $\frac{n}{5850}\left(\frac{h}{10}\right)^5$, $\frac{n}{2400}\left(\frac{h}{10}\right)^5$, $\frac{n}{9600}\left(\frac{h}{10}\right)^5$.

The stress on the bolts$=2n\left(\frac{h}{12}\right)^2\times 35=50n\left(\frac{h}{10}\right)^2$. If, for example, the girder is 24″ deep, n=3′-0″, and we use 2″x4″ rough cleats, flat, $D=\frac{3}{2400}\times 2.4^5=0.1$ inch, or if the girder is six feet deep and we concrete so rapidly that there may be 40″ of wet concrete in the girder, and n=2′ and use 2″x″ on edge, then $D=\frac{2x4^2}{9600}=0.214$ inches, and the strain on the bolts in the latter case$=50\times 2\times 4^2=1600$ pounds, which will correspond to ½″ bolts.

The data for column forms are given on page 102, and shown in Figure 24 on page 98. The forms were figured for a pressure of wet concrete 12′ in height, and we assume that the concrete pressure=water pressure.

We ascertain the depth of wet concrete producing a pressure =water pressure in a column or wall, by forcing a stick of wood, 1″x1″, into the concrete. The greatest depth it can penetrate gives us the depth of wet concrete in above sense.

The boards are held together by frames in distances of 2′ 0″, and the pressure per side of frame is given in eighth column, The frames consist of 2″x4″ or 2″x6″, nailed directly to the sides, and are held together by bolts in one direction and by 1″ cleats, nailed to the 2″x4″, as shown in Figure 25. The boards for the sides of the columns should be surfaced one side and both edges, and the deflection of the boards between the frames will be found to be less than 1/30″ For columns larger than 27″ the frames should consist of 4″x4″, bolted in four directions. In this case the side boards are cleated together by 1″x4″ every four feet. The corners should be rounded off by nailing right angle triangular strips in the corner of the forms, either of 1″ or 2″ sides. On one side a loose board should be left at the bottom to allow of cleaning out of the column form before concreting.

The octagonal column forms of an inscribed circle of 13.5″, 18.5″, 23.3″ and 28″ diameter can be built, using stock lumber as given in the table and filling in the spaces between the planks by one inch triangular strips. The frames to hold the planks together can be made of the same size as given for the corresponding square columns.

Form lumber per lineal foot, as given in the table, includes the bracing to hold the forms in a vertical position.

Where the length of the form does not correspond to the commercial length of lumber, there will be a certain waste, but it will rarely exceed 10% of the figure given in the table.

Figures 27 to 32 show the scheme of the wall forms which we recommend. Where the progress of the concrete is thus that not more than three, five or eight feet of wall are concreted in two or three hours, the forms should be built according to Figures 27, 28, Figures 29, 30, or Figures 31, 32, respectively.

These forms are figured for a height of wet concrete of 3′, 5′ and 8′. It is generally cheaper to use wires to hold the forms together, when the wires can be tightened from the inside, but for thinner walls the use of bolts will be found considerably cheaper and more reliable, although it takes more lumber.

For uprights 1-⅝"x3⅝" may be used in all cases.

The bolts in all cases should not be less than ⅝".

In Figure 28 the bolts should be spaced 7' c.c., when full 2"x4" are used for whaling pieces, or 6'-0" when 1-⅝"x3-⅝" are used.

In Figure 30 the bolts must be spaced 5'-0" c.c., or 4'-0" c.c., according as to whether 2"x4", or 1-⅝"x3-⅝" are used for whaling pieces.

In Figure 32 only one 2"x4" need to be used when the bolts are placed next to the uprights.

The following table gives the quantities of lumber per square foot in the various types of wall centering also the number and weight of bolts and wires.

	Feet of Form Lumber per Square Foot of Wall (Both Sides are Included)	Weight of Wire per Square Foot of Wall 17" Thick	Weight of Bolts per Square Foot of Wall 6" Thick	
			2x4 Whaling	1⅝x3⅝
Figure 27	2.75	.056		
Figure 29	2.80	.135		
Figure 31	2.90	.23		
Figure 28	3.65		.135	.16
Figure 30	3.90		.23	.28
Figure 32	3.45			.46

The deflection in the boards are less than 1/30", and including the deflection of the 1⅝x3⅝ uprights and whaling pieces, the greatest spread of the forms should not exceed 1/10" at each side, when above arrangement is used.

The table on page 99 gives the best arrangement of joists, girders and supports for the typical floor constructions of pages 44-54. It is always assumed that 1″ flooring and 2″ planks for beam forms are used. The size of the joists is given in the third column; the exact spacing of the joists, however, should be found by consulting tables on page 92. A † denotes that the sizes must be full, or larger joists should be used. The size of the girders, supporting the joists, is given in the fourth column, and in nearly all cases full sizes should be used. The number of supports is given in the fifth column and a * denotes that the supports should be braced in both directions at mid-height by 1″x6″ boards. The number of supports were found by figuring the weight of the whole panel and dividing by the permissible load on one support, and making a proper deduction for the weight carried directly by the columns. In the sixth column is given the serial number of the sketches on pages 100 to 101, showing the arrangement of the form work. The black circles in these figures indicate the concrete columns; the black lines, the beams and girders; the dotted lines the 3″x6″ or 3″x8″ girder; the plain circles one 4″x4″ support, and the circles with a cross denote double 4″x4″, as shown in Figure 34. The direction of the joists is indicated by arrows. In some cases the load on one or two supports in one panel is greater than the average, and it may be of advantage to use for these supports 4″x6″ or 6″x6″.

For ceiling heights greater than twenty feet it is generally cheaper to use 6″x6″ supports, instead of the 4″x4″ given in the tables. For example, where the length of the 6″x6″ supports are 22′ it takes the same number of supports as where the length of 4″x4″ are 10′. To facilitate the leveling up of the floor and the taking down of the forms, each support should be set on two wedges, each being 12″ long and 4″ wide (or wider for supports larger than 4″x4″) and ½″ thick at one end and 2½″ thick at the other end. It takes as a rule one wedge for every 20 square feet of floor.

The cost of the form work can be very materially diminished, if working drawings for each beam and girder form, wall panels, etc., is made, and the presence of a competent engineer or draftsman will easily save 10 to 30% of the carpenter payroll.

NAILS.

Common wire nails have the following lengths:

	2d	3d	4d	5d	6d	8d	10d	12d	16d	20d
Length in Inches......	1	1¼	1½	1¾	2	2½	3	3¼	3½	4
Numbea in 1 Pound....	716	440	300	210	163	93	66	50	40	32

To nail down the one inch flooring, which consists generally of boards 6″ to 10″ wide, not more than three, at the utmost five 4d. nails should be used in one length of board, or it takes in an average, including all waste, not more than 0.25 pounds of 4d. per 100 square feet of floors.

It takes about one 4d. nail per lineal foot of each beam or girder to fasten the corner strips, and about four 6d. nails to fasten the ledger boards, and about one and one-third 16d. nails per lineal foot to fasten the bottom to the sides.

In a floor construction as given for the first floor on page 83, we have per 100 square foot in an average 0.5 pounds of 4d., 1.5 pounds of 6d. nails, and 0.4 pounds of 16d. nails.

RETAINING WALLS.

The earth pressure per lineal foot of a wall of the height h in feet we assume to be given by the formula $P = \frac{1}{2} s h^2 tg^2 (45° - \frac{a}{2})$, when s the weight per cubic foot of the earth and a the angle of repose. Let $s = 100$ pounds and $a = 32°$, then $P = 15\ h^2$, and by differentiation we obtain the pressure per square foot at a point h below the wall as $p = 30\ h$, showing that the pressure is distributed along the height of the wall similar to water pressure, and that the resultant of the earth pressure must act at a point, which is $\frac{2}{3}$ h below the top. Neglecting the friction of the earth against the wall, or in other words assuming the pressure to act horizontally, this earth pressure produces an overturning moment about the center of the base of the wall$= 15h^2 \times \frac{1}{3}h = 5h^3$ In all retaining walls in the tables the base$= \frac{1}{2}h$, the toe$=$ $\frac{1}{8}h$ and the heel$= \frac{3}{8}h$. To simplify the calculation we shall assume that the concrete per cubic foot does not weigh more than the earth, then we can readily put the moment from the weight of the earth about the center of the base $= 100 \times \frac{3}{8}h \times h \times \frac{1}{16} \times 100 = 2.34h^3$, which moment acts in opposite direction of that of the earth pressure. Deducting, we have the resultant moment$= 2.66h^3$. Applying Hooke's law, this bending moment must be equal to the moment of resistance of the rectangle of the base, which is one foot wide and $\frac{1}{2}h$ deep multiplied by the extreme stress at the edges per square foot Or $2.66h^3 = S \times \frac{1}{6} \times (\frac{1}{2}h)^2$ or the extreme stress per square foot $S = 64h$, which means a compression at the toe and an ideal tension at the heel. To these stresses must be added the average uniform pressure produced by the weight of the earth$= 100 \times \frac{3}{8}h^2 \div \frac{1}{2}h = 75h$, which results in a distribution of pressures as shown by the trapezoid abcd in Figure 57 In as much as there are only compressive stresses produced at the base, the wall is perfectly stable against overturning It is also safe against sliding, as the relation between the earth pressure and the weight resting on the base of the wall

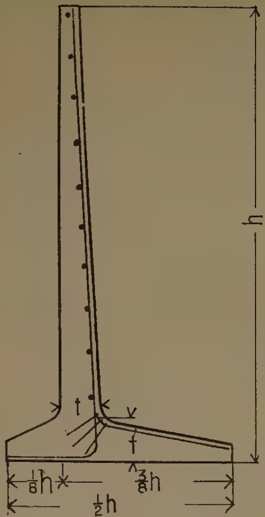


Fig. 55. Retaining Wall, Type "A."

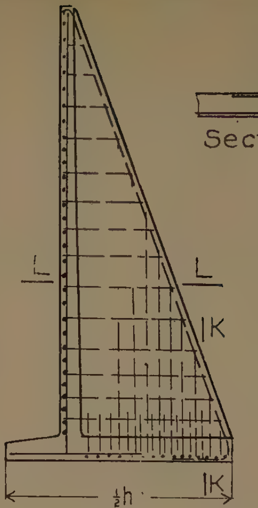
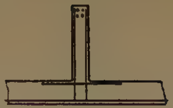
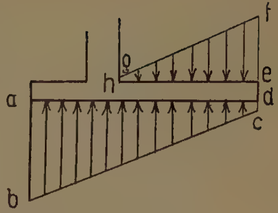


Fig. 56. Retaining Wall with Counterforts.





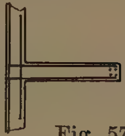


Fig. 57. Distribution of Stresses on Base.

ab=139h.
dc=75h—64h=11h.
fe=100h—11h=89h.

is as $15h^2:37.5h^2=0.4$, while the coefficient of friction is about 0.566.

The toe may be lengthened and the heel shortened till they are both of the same length, without jeopardizing the safety of the wall against overturning, but as the weight on the base becomes smaller the safety against sliding is diminished, and we have to resort either to an inclination of the base or to projections into the ground from the base, in order to increase the sliding resistance.

We can clearly see that the upward reaction of the ground stresses the toe like a cantilever. In case of the heel we have to consider that the reaction of the ground is smaller than the weight of the earth above it, therefore, a reversal of stress takes place, and the distribution of the downward pressure is given by the trapezoid efgh in Figure 57.

The stresses are only immaterially altered if the real weight of the concrete enters the calculations, for the reason that the thickness of concrete is small in comparison to the dimensions of the structure.

The retaining walls as Fig. 55 are figured on the cantilever principle both for the toe and the heel, and on account of the reversal of the stresses the vertical rods of the wall must run into the upper face of the heel and into the bottom of the toe. The toe takes up 20% of the overturning moment, and the heel 80%, and in the same ratio the steel rods should be divided.

To prevent cracks it is the writer's practice to reinforce the wall horizontally at the top by ½% and diminishing this percentage to ⅛ or ¼ at the base of the wall. The writer does not place any longitudinal reinforcement in the toe or heel. Place three ¼" stirrups at each rod, turning into the heel, as shown in Figure 55.

Up to a height of 15 feet this type of walls are cheaper than reinforced concrete walls with counterforts, as shown in Figure 56. The bending moments in the face of these walls are figured by the formula $\frac{pl^2}{12}$, when $p=30h$, and $l=$ clear distance between counterforts in feet. The toe is again a cantilever, while the heel is stressed the most at the edge by a uniform load per square foot=89h. Also here the bending moment is taken as $\frac{pl^2}{12}$

The table gives the reinforcement per lineal foot of toe and the reinforcement for the heel plate at the edge, which reinforcement may be safely diminished to ¼% at the point h. The main reinforcement in the counterfort is obtained by considering the counterfort as a slab of the width given and a depth=0.35h and is also given in the ninth column, while the horizontal and vertical bars marked "a" and "b" in Figure 56 must be figured to take up the earth pressure=30h × distance of counterforts and the weight of the earth according to diagram Figure No. 57.

To take care of the negative moments both in the wall and in the heel plates, rods of a length=0.4 the distance between counterforts must be placed at the counterforts both in wall and heel slabs.

The main reinforcements of the counterforts should extend into the heel plate at least 1/10 h, and ½ of the rods may be stopped of at about ½ the height of the wall. The reinforcement of the toe should be 0.29 h long. The horizontal reinforcement of the wall may be gradually diminished from the values given for the base to the value given for the top. As vertical reinforcement, ½ round 12″ c. c. or its equivalent will suffice.

For a surcharge equal to the weight of earth of the height h^1, the pressure per foot of wall$=15\ (h^2+2hh^1)$ and the distance of the resultant earth pressure from the base $=\frac{h}{3}\ \frac{3h^1+h}{2h^1+h}$

PROPERTIES OF RETAINING WALLS.

TYPE "A"									FOR BASEMENTS	
Height of Wall from Base to top of Fill	Overturning Moment Center of Base Foot-lbs.	Width of Base	Thickness of Wall		Area of "ertical Reinforcement per lineal foot Sq. Ins.	Cubic feet of Concrete per lineal foot	Total weight of Reinforcement per lineal foot	Form Lumber per lineal foot Feet Board Measure	Thickness of Wall Inches	Area of vertical Reinforcement per lineal foot Sq. Inches
			at the Base Inches	at the Top Inches						
4	320	2′–0″	4″	4	.12	2.00	6.0	16	4	.120
5	625	2′–6″	4″	4	.12	2.50	7.5	20	4	.120
6	1080	3′–0″	4½″	4	.20	3.20	10.4	24	4	.120
7	1720	3′–6″	6″	4	.20	4.40	14.2	28	4	.120
8	2560	4′–0″	7″	4	.25	5.50	17.3	32	5	.150
9	3645	4′–6″	8″	5	.33	7.30	28.2	36	6	.180
10	5000	5′–0″	10″	5	.33	9.40	33.0	40	6	.280
11	6650	5′–6″	11″	5	.48	11.00	40.5	44	7	.290
12	8640	6′–0″	12″	5	.58	12.70	50.0	48	8	.336
13	11000	6′–6″	15″	6	.50	17.0	51.0	52	8	.528
14	13700	7′–0″	15″	6	.75	18.40	69.0	56	9	.540
15	17000	7′–6″	18″	6	.66	22.5	73.0	60	10	.600
16	20500	8′–0″	20″	6	.72	26.0	83.0	64	12	.576
17	24500	8′–6″	22″	6	.80	30.0	95.0	68	12	.792
18	29200	9′–0″	24″	6	.85	34.0	106.0	72	15	.630

The retaining walls for basements are figured as a slab, held top and bottom, by first floor and basement floor, respectively; the bending moment per lineal foot $= 15h^2 \times \frac{h}{8} =$ about $2h^3$. Longitudinal reinforcement equals ¼%.

RETAINING WALLS WITH COUNTERFORTS.

Height of Wall from Base to Top of Fill in Feet	Overturning Moment Center of Base Foot-Pounds	Thickness of Wall		Horizontal Reinforcement in Wall per Foot		COUNTERFORTS			BASE				Unit Quantities per lineal foot of Wall		
									Toe		Heel				
		At Base	At Top	At Base Square Inches	At Top Square Inches	Distance Center to Center	Thickness	Sectional Area of Main Reinforcement in Square Inches	Thickness	Sectional Area of Reinforcement per Lineal foot of Wall	Thickness	Sectional Area of Reinforcement at Point Farthest from Wall	Concrete Cubic Feet	Steel Pounds	Lumber for Forms Board Measure
18	29200	9	6	.36	.36	10′–8″	8″	1.22	12″	.36	12″	.66	23.6	83.2	92
19	34200	9	6	.36	.36	10′–8″	8″	1.34	12″	.42	12″	.78	25.4	93.	101
20	40000	9	6	.38	.36	10′–8″	8″	1.44	12″	.60	12″	.86	27.3	105.	110
22	53300	10	6	.36	.36	11′	12″	2.34	12″	.90	12″	1.00	32.7	131.	125
24	69100	10	6	.42	.36	11′	12″	2.52	15″	.75	15″	.75	40.5	143.	140
26	88000	11	6	.40	.36	11′	12″	3.97	15″	1.08	15″	.80	45.6	169.	150
28	110000	11	6	.55	.36	11′	12″	4.90	15″	1.62	15″	.90	50.3	221.	165
30	135000	12	6	.38	.36	11′	12″	5.20	15″	2.00	15″	1.00	56.2	238.	180
35	215000	12	6	.51	.36	11′–3″	15″	7.50	24″	1.40	24″	.72	82.0	267.	225
40	320000	12	6	.72	.36	11′–3″	15″	9.60	27″	2.35	27″	.81	106.0	395.	285
50	625000	15	6	.54	.36	11′–6″	18″	16.20	36″	3.55	36″	1.08	179.0	621.	400

These walls are just as safe against overturning as solid concrete walls of a thickness=0.45 h.

ROUND TANKS.

In designing a tank of a given capacity, where the diameter or the height is not fixed by some special requirements, the first question which arises is to find the diameter and height of the tank which will make the cost a minimum.

Let "a" be the average cost per square foot of the cylindrical shell and "b" the average cost per square foot of excavation, bottom, covering (if any), fill (if any), then we can find by the theory of maxima and minima for the minimum cost the important relation $H = \frac{1}{2}D \times \frac{b}{a}$, when H= height and D=diameter of tank in feet.

The cost per square of the shell varies from about 35c for the very small tanks to about 80c for the largest tanks, while the cost of the bottom does not vary more than from 13 to 17c per square foot, and that of the covering from 30 to 50c per square foot.

After one or two preliminary trials, in which the capacity table on page 122 will be found helpful, the proper relation of H to D will be found, and by interpolation the proper H and D may be taken from the same tables.

The amount of reinforcing in square inches per lineal foot of the shell is at a point h below the high water line given by $62.5h \times \frac{1}{2}D \div 16000 =$ about $\frac{hD}{500}$, 16000 pounds being the allowable fibre stress on the steel.

In the tables on pages 120 and 121 the second column gives the required reinforcement for values Dh up to 5000. The reinforcement per lineal foot should, however, never be reduced below ¼% of the concrete section, even if the tables give for the upper portion of the tanks smaller values. For convenience of ready estimating the cost, the third, fifth and sixth columns on pages 120 and 121 give the total amount of steel, concrete and form lumber for the cylindrical sides for values of D × greatest height up to 5000, and the ninth and tenth column on page 122 give the total of concrete and steel in the bottom, and on page 123 will be found the data for dome coverings.

The vertical reinforcement in the shells was assumd $=\frac{1}{4}\%$. The rods in the bottom must run in both directions and extend at least a foot into the shells.

For square or rectangular tanks the minimum cost is obtained when $H=\frac{1}{2}D\times\frac{b}{a}$ when D the side of the tank in feet. For rectangular tanks a similar proportion will apply. The bottoms in square tanks can be made of the thickness given for round tanks, and the sides can be taken from table below. For the covering a girderless floor construction, as given in tables on page 55, will be found safer and cheaper than the groined arch construction which is generally employed in the United States.

RESERVOIR WALLS.

Depth of Wall in Feet	Thickness in Inches	Area of Vertical Reinforcement per Lineal Foot for		Depth of Wall in Feet	Thickness in Inches	Area of Vertical Reinforcement per Lineal Foot for	
		Water Pressure	Water And Earth Pressure			Water Pressure	Water And Earth Pressure
6	4 "	.15	.12	14	14″	.60	.42
7	4½″	.27	.135	15	15″	.65	.45
8	5½″	.33	.165	16	16″	.86	.48
9	6½″	.39	.195	17	18″	.86	.54
10	7½″	.45	.225	18	20″	.86	.60
11	9 "	.49	.270	19	22″	.93	.66
12	10 "	.57	.300	20	24″	.93	.72
13	12 "	.58	.360				

The bending moment for water pressure on one side $=\frac{62.5}{2}h^2 x \frac{h}{8}=3.9h^3$.

The bending moment for earth pressure on one side $=15h^2 x \frac{h}{8}=1.88h^3$.

For water pressure one side and earth pressure the other side $=2.02h^3$.

PROPERTIES OF

Product H' x D'	Sectional Area of Reinforcement per foot at point h' x d'	Total Weight of Reinforcement in Shell for H' x D'	Thickness of Shell for H' x D'	Total Amount of Concrete in Shell	Total Amount of Lumber Required for Forms in 1000 Ft. B. M.
10	.06	16	2"	5	.11
20	.06	31	2"	10	.22
30	.08	62	2½"	20	.33
40	.09	93	3"	31	.44
50	.10	120	3"	40	.55
60	.12	150	3"	47	.66
70	.14	185	3"	55	.77
80	.16	248	3½"	73	.88
100	.20	365	4"	104	.99
110	.22	415	4"	115	1.21
120	.24	468	4"	125	1.32
130	.26	525	4"	136	1.43
140	.28	584	4"	146	1.54
150	.30	643	4"	157	1.65
160	.32	710	4"	167	1.76
170	.34	775	4"	178	1.87
180	.36	950	5"	237	1.98
190	.38	1030	5"	250	2.09
200	.40	1100	5"	263	2.20
225	.45	1320	5"	295	2.47
250	.50	1530	5"	329	2.75
275	.55	1790	5"	361	3.02
300	.60	2210	6"	471	3.30
325	.65	2510	6"	510	3.57
350	.70	2820	6"	550	3.85
375	.75	3150	6"	590	4.10
400	.80	3500	6"	630	4.40
450	.90	4200	6"	705	4.95
500	1.00	4800	6"	785	5.50
550	1.10	5850	6"	865	6.05
600	1.20	6800	6"	945	6.60
650	1.30	7500	6"	1020	7.15

CIRCULAR TANKS.

Product H' x D'	Sectional Area of Reinforcement per Foot at Point h' x D'	Total Weight of Reinforcement in Shell for H' x D'	Thickness of Shell for H' x D'	Total Amount of Concrete in Shell	Total Amount of Lumber Required for Forms in 1000 Ft. B. M.
700	1.40	9200	7″	1280	7.70
750	1.50	10100	7″	1370	8.25
800	1.60	11600	7″	1470	8.80
850	1.70	12800	7″	1560	9.35
900	1.80	14100	7″	1650	9.90
950	1.90	15600	7″	1740	10.45
1000	2.00	17000	7″	1830	11.00
1100	2.20	20200	7″	2020	12.10
1200	2.40	23500	7″	2200	13.20
1300	2.60	27000	7″	2380	14.30
1400	2.80	31000	7″	2570	15.40
1500	3.00	35200	7″	2750	16.50
1600	3.20	39700	7″	2930	17.60
1700	3.40	44500	7″	3110	18.70
1800	3.60	49800	8″	3770	19.80
1900	3.80	54800	8″	4000	20.90
2000	4.00	60500	8″	4200	22.00
2100	4.20	65800	8″	4400	23.20
2200	4.40	73000	8″	4600	24.20
2300	4.60	79000	8″	4820	25.30
2400	4.80	85600	9″	5650	26.4
2500	5.00	94000	9″	5900	27.5
2600	5.20	101000	9″	6150	28.6
2700	5.40	108000	9″	6350	29.7
2800	5.60	118000	10″	7300	30.8
2900	5.80	122000	10″	7550	31.9
3000	6.00	134000	10″	7820	33.0
3300	6.60	162000	11″	9500	36.3
3700	7.40	202000	11″	10700	40.7
4000	8.00	235000	12″	12500	44.0
4500	9.00	289000	13″	15300	49.5
5000	10.00	370000	15″	19500	55.00

CONTENTS OF CIRCULAR TANKS AND DATA ON BOTTOMS.

Diameter in Feet D	CAPACITY IN 1000 GALLONS					Thickness of Bottom in Inches	Weight of Steel in Bottom per Square Foot	Total Concrete in Bottom Cubic Feet	Total Steel in Bottom Pounds
	One Foot Deep	$\frac{D}{3}$ Deep	$\frac{D}{2}$ Deep	¾D Deep	D Deep				
10	.583	1.96	2.91	4.40	5.81	4″	.96	26	76
11	.710	2.60	3.92	5.85	7.75	4″	.96	32	91
12	.849	3.37	5.05	7.50	10.10	4″	.96	38	109
13	1.00	4.31	6.43	9.60	12.90	4″	.96	45	128
14	1.156	5.35	8.05	12.10	16.10	4″	.96	51	148
15	1.33	6.68	9.86	14.85	19.80	4″	.96	59	170
16	1.51	8.00	12.00	18.00	24.00	4″	.96	67	194
17	1.70	9.59	14.40	21.7	28.80	4″	.96	76	218
18	1.91	11.40	17.10	25.8	34.20	4″	.96	85	245
19	2.12	13.40	20.00	30.2	40.00	4″	.96	95	273
20	2.35	15.60	23.40	35.2	46.90	4″	.96	105	301
22	2.85	20.90	31.25	47.0	62.50	5″	1.2	159	456
24	3.39	27.00	40.5	61.0	81.0	5″	1.2	189	543
26	3.98	34.30	51.5	77.0	103.0	5″	1.2	222	639
28	4.61	43.00	64.3	96.5	128.5	5″	1.2	257	740
30	5.29	52.90	79.00	118.5	158.0	5″	1.2	295	848
32	6.01	64.10	95.8	144.0	192.0	6″	1.44	403	1160
34	6.79	76.50	115.0	172.0	230.0	6″	1.44	455	1310
36	7.63	91.10	136.8	205.0	274.0	6″	1.44	509	1470
38	8.49	107.50	162.0	241.0	324.0	6″	1.44	568	1640
40	9.36	125.00	188.0	282.0	375.0	6″	1.44	629	1810
45	11.90	178.00	267.0	400.	534.	6″	1.44	797	2290
50	14.70	244.00	365.0	550.	730.	6″	1.44	982	2830
55	17.80	325.00	488.0	730.	978.	6″	1.44	1190	3430
60	21.10	421.00	634.0	945.	1270.	6″	1.44	1420	4070
65	24.80	537.00	805.0	1210.	1610.	6″	1.44	1660	4780
70	28.80	669.00	1000.0	1500.	2000.	6″	1.44	1930	5550
75	33.0	824.00	1240.0	1860.	2470.	6″	1.44	2210	6380
80	37.6	1000.00	1500.0	2250.	3000.	6″	1.44	2510	7250
85	42.5	1200.00	1800.0	2700.	3600.	6″	1.44	2840	8200
90	47.5	1420.00	2140.0	3210.	4280.	6″	1.44	3180	9200
95	53.0	1670.00	2500.0	3770.	5000.	6″	1.44	3540	10200
100	58.8	1960.00	2900.0	4400.	5800.	6″	1.44	3930	11400

1 cubic foot=7.48 U. S. gallons.

PROPERTIES OF DOMES

	Total Load 300 Pounds per Square Foot					Total Load 600 Pounds per Square Foot				
Diameter of Dome Feet	Stress in Bottom Ring 1000 Pounds	Thickness of Concrete at the Base	Weight of Reinforcement per Sq. Ft.	Total Concrete in Dome Cubic Feet	Total Steel in Dome Pounds	Stress in Bottom Ring 1000 Pounds	Thickness of Concrete at Base	Weight of Reinforcement per Sq. Ft.	Total Concrete in Dome Cubic Feet	Total Steel in Dome Pounds
15	20.3	2	1.73	31	320	22.5	2″	.48	33	94
20	36.0	2	2.28	55	750	40.0	3″	.72	87	250
25	56.2	2	2.73	86	1280	62.5	3½	.84	160	456
30	81.0	2½	3.30	154	2440	90.0	4	.96	261	751
35	110.0	3	3.87	250	3880	122.5	5	1.20	300	1280
40	144.0	3½	4.44	385	5800	160.0	5½	1.32	432	1840
45	183.0	4	5.01	552	8300	202.5	6	1.44	590	2540
50	225.0	4½	5.58	720	11400	250.0	7	1.68	845	3650
	Total Load 900 Pounds per Square Foot					Total Load 1200 Pounds per Square Foot				
15	33.8	3	.72	49	141	45.0	4	.96	65	188
20	60.0	4	.96	116	333	80.0	5½	1.32	106	460
25	94.0	5	1.20	151	651	125.	6½	1.56	196	850
30	135.0	6	1.44	264	1130	180.	8	1.92	347	1510
35	184.0	7	1.68	415	1790	245.	9½	2.28	560	2440
40	240.0	8	1.92	615	2670	320.	10½	2.53	811	3540
45	305.0	9	2.16	881	3810	405.	12	2.88	1170	5100
50	375.0	10	2.40	1200	5210	500.	13½	3.24	1630	7020
	Total Load 1500 Pounds per Square Foot					Total Load 1800 Pounds per Square Foot				
15	56.3	5	1.20	55	235	67.5	6	1.44	66	282
20	100.	6½	1.57	126	544	120.	8	1.92	154	665
25	156.	8½	2.04	257	1110	187.5	10	2.40	300	1310
30	225.	10	2.40	435	1890	270.	12	2.88	521	2260
35	306.	11½	2.76	680	2950	367.5	14	3.36	829	3600
40	400.	13½	3.25	1040	4520	480.	16	3.84	1225	5350
	Total Load 2100 Pounds per Square Foot					Total Load 2400 Pounds per Square Foot				
20	140.	9	2.16	174	750	160.	10½	2.52	202	880
30	315.	14	3.36	610	2650	360.	16	3.85	690	3030
40	560.	18½	4.45	1440	6200	640.	21½	5.16	1670	7200
50	875.	23½	5.62	2850	12200	1000.	27	6.50	3250	14100

In domes for 300 pounds per square foot the weight of reinforcement in base ring is included in weight of the steel.

ELEVATED TANKS.

Capacity in 1000 U. S. Gallons	Diameter and Height of Tank in Feet D′ x H′	Diameter of Base Ring d′	Height of Bottom of Tank Above Ground h′	Supporting Columns		Size of Footings	Thickness of Supporting Shell	Total Concrete Cubic Feet	Total Steel Pounds	Total Form Lumber in Feet B. M.
				Number	Size					
50	24 x 15	18	20	6	17″ x 17″	4′–6″ square		1350	8280	11500
75	27 x 18	21	40	6	23″ x 23″	5′–3″ "		2555	16460	19100
100	30 x 20	22	60	6	29″ x 29″	6′–0″ "		4480	23200	27600
200	38 x 24	28	80			7′–0″ circular	6″	7495	38850	52000
300	44 x 27	33	100			9′–6″ "	8″	12505	65450	70700
400	48 x 30	36	100			11′ "	9½″	15930	88300	81500
500	54 x 30	40	100			12′ "	11″	21275	110900	94600
600	57 x 32	43	100			13′ "	12″	25630	131200	102000
700	60 x 34	45	100			14′ "	13″	29680	154500	116000
800	60 x 39	45	100			16′ "	14″	33500	190200	121500
1000	65 x 41	49	100			18′ "	16″	44350	235500	130200
1500	70 x 53	53	100			20′ "	24″	66200	396000	153000

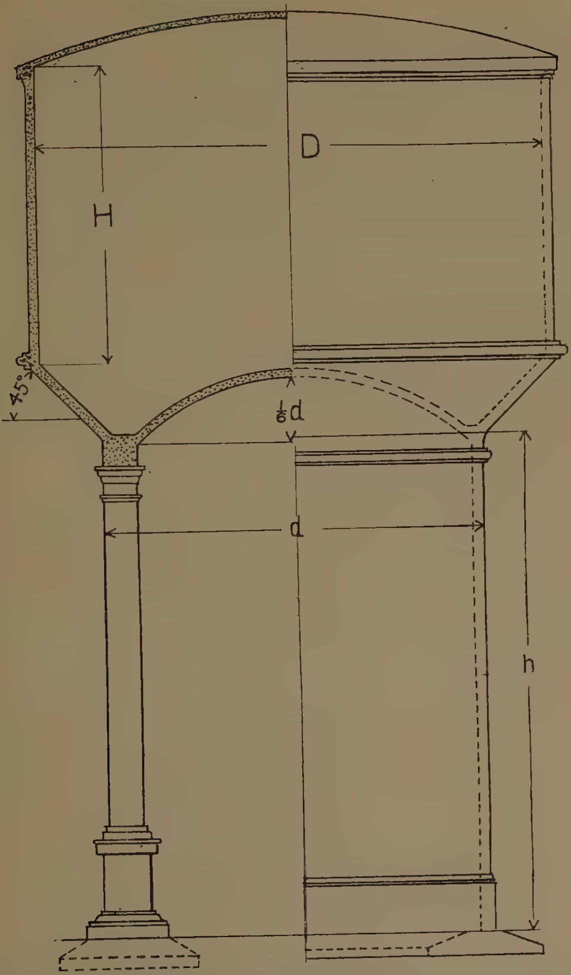


Fig. 58. Half Section and Half Elevation of Elevated Water Tanks.

In this case the reinforcements given in the fourth column must be placed at both sides of the wall.

Longitudinal reinforcement should be ¼%.

Example: To build a covered reservoir of 300,000 gallons capacity. It is evident that it will be economy to use portion of the excavation for backfill, and therefore the reservoir should be built partially under and partially above the natural ground. At a first trial it will be natural to assume that the height should be as little as possible, say one-quarter of the diameter. The table on page 122 gives the corresponding diameter to 60′. Figuring the concrete at 25c per cubic foot, steel at 2½c per pound, forms at $45 per 1000 feet B. M., excavation at 50c, fill at 30c per yard, we find that the relation b:a=nearly two, or that H should be nearly equal D. On the other hand, a tank being as high as the diameter, of the required capacity would be, according to the tables, 38′ in diameter and rather inconvenient to build and to cover with dirt, and we shall settle on a tank 50′ in diameter and 20′ high, being ten feet in the ground. The dirt from the excavation will be just sufficient to cover the top three feet deep and to form an embankment around the sides. D×h at the lowest point=1000, at a point 5′ higher up=750, at middle height=500, at the quarter point=250, requiring, according to tables on pages 120 and 121, 2.00, 1.50 and 0.50 square inches of steel per lineal foot, or 1″ square bars, spaced 6″, 8″, and ⅝″ square bars spaced 9″ respectively. The reinforcement at the top of the cylindrical shell should not be less than ¼% of the section$=\frac{7\text{x}12}{400}=.21$ square inches. As vertical reinforcement we shall use ¾″ square bars 30″ c. c.=¼% of the concrete section.

The bottom, according to the table on page 122, will be made 6″ thick, and reinforced in each direction by ¼%=.18 square inches, or ⅜″ square bars 9″ c. c.

The dome will have a rise of 5′ and be 4½″ thick and reinforced by ¼% in two directions, or $\frac{4.5\text{x}12}{400}=.135$ square inches per lineal foot. The thrust of the dome must be taken up by a circular reinforcement at the base of the dome. The required sectional area is found by dividing the stress given in the table by 16000, or $\frac{225.000}{16.000}$=14 square inches, corresponding to fourteen 1″ square bars.

The reinforcement in the shell should overlap at least 50 diameters, and the laps should be staggered.

The simplest way to make the shell waterproof is to give it, on the inside, three coats of cement finish, each about ⅛″ thick, and each applied before the preceding coat is set. Before applying the finish the surface must be thoroughly cleaned and wetted. The finish is to consist of one part cement, 1/10 part hydrated lime, and 1 part sharp sand.

The entire tank requires 3540 cubic feet of concrete, 32400 pounds of steel, and 19 M feet of lumber.

As a comparison we shall figure the quantities for a square tank of the same capacity. Let us assume the tank is 50′ square and 18′ deep, allowance of 2′ being made for air space, which in case of a dome cover is obtained by the rise of the dome. In as much as the water stands only 16′ feet high and the wall is 18′ high, we shall take the dimensions for a 17′ wall given in table on page 119 for the side walls, which means that we have per square foot 1.5 cubic feet of concrete, 6 pounds of steel, and 4 feet of lumber for forms. For the roof construction we shall adopt a girderless floor, being supported by columns 16′ feet on centers, which floor construction requires .625 cubic feet of concrete, 3.84 pounds of steel and 3.00 feet of lumber per square foot. The columns we shall make 15x15¼, reinforced by four ⅞″ round bars.

This tank requires 8250 cubic feet of concrete, 33300 pounds of steel and 22700 feet of lumber, and although the unit labor might be a trifle less in this case than for a round tank, the latter will be found cheaper than the square tank.

The table on page 123 gives data for the design of domes, which are uniformly loaded. The rise of the domes was assumed =1/10 of the diameter in case of a total load of 300 pounds per square foot, and=⅙ of the diameter in all other instances. Concrete domes are figured for shear only, because the shear is not much less than the compression per lineal foot, but the allowable shearing stresses are considerable less than the stresses which may be allowed in compression.

The vertical reaction per lineal foot of circumference at the base of the dome$=\frac{3.14pD^2}{4} \div 3.14D = \frac{pD}{4}$ and this force produces a radial tension of $2.4 \times \frac{pD}{4}$ and $\frac{4}{3}\frac{pD}{4}$ for domes of a rise $=\frac{1}{10}D$ and $\frac{1}{6}D$ respectively, which radial forces multiplied by half the diameter$=0.6pD^2$ and $.333pD^2$ gives us the stresses in the base ring. The shear per lineal foot of circumference in radial direction$=0.231pD$ and $0.20pD$ respectively, and a shearing stress of 75 pounds per square foot was allowed in figuring the tables. There is no necessity to reinforce the dome in circumferential and radial directions, and it is the writer's practice to place a reinforcement$=$¼% in two main directions under right angles to take care of possible unequal loading.

The thickness of the domes at the base may be safely decreased to ½ at the crown, which was taken in consideration when figuring the concrete, in all cases where thickness at the base is given at 5″ or larger.

The elevated tanks, as detailed in table on page 124 and as shown in Figure 58, will be found in most cases cheaper than standpipes or other elevated tanks, and are decidedly of a sightlier appearance. The bottom consists of a portion of a sphere and a truncated cone, and if the relative dimensions of the cone and dome are adopted as given in the tables and shown in Figure 58, there is no perceptible thrust on the base ring.

The smaller tanks may be supported on columns, but the larger tanks should be supported by a reinforced concrete shell, of a thickness as shown in table, and reinforced by ¼% in both vertical and circumferential direction. The permissible load on the ground was assumed=5000 pounds per square foot, and the wind pressure was assumed=30 pounds per square foot for a flat, or 20 pounds for the projection of a round surface.

Where the tank is supported by columns a stiffening ring of a section=section of the supporting columns should be adopted to transmit the loads to the columns. The reinforcement of this ring for the three cases in the tables by four 1″ round bars will suffice. Extra rods should be provided for the negative bending moments over the supports. At the junction of the conical bottom with the cylindrical shell is acting a tension = weight of shell and cover divided by 3.14, which must be taken care of by a circular reinforcement.

The thickness of the conical bottom=thickness of the shell, and the circular reinforcement can be taken from pages 120 and 121 for the proper values of $d \times h$.

GRAIN ELEVATORS.

We assume the weight of grain=50 pounds per cubic foot, and the angle of repose=29°. Then by common theory of earth pressure the pressure per lineal foot on a wall h feet high=$7.7h^2$, and the pressure per square foot at h below the surface=15.4h. This pressure in a bin of the width s in feet increases, however, only until h=s cot. 29°=1.8s. Below this depth the friction against the sides equals the additional weight of the grain, and no matter how deep the bin the pressure never exceeds $15.4\times 1.8s=27.75s$, a fact fully demonstrated by practical experience.

The bending moments caused by a uniform load p per square foot in a square frame of a side s in feet are given by the formula $\frac{ps^2}{24}$ and $\frac{ps^2}{12}$ for the center of the sides and the corners respectively, or, as p=27.75s, the bending moments= $1.15s^3$ and $2.3s^3$ respectively.

It will be found that the bin walls given on page 133 were figured according to these bending moments. The bottom of these bins is generally built as a truncated pyramid, which is suspended from the sides. The weight on the bottom is not greater than the weight of the grain of a depth = s′. The total weight on the bottom divided by the girt of bin gives the vertical force per lineal foot, which must be taken care of by the suspension rods "r,", the section of which is obtained by dividing the stress given in the seventh column by 16000. These rods should extend into the bin walls for a length = s′ and each rod must be anchored by a stirrup at least 2′ long into the adjoining bottom, in order to take care of the horizontal component of the pull from these suspension rods.

The horizontal reinforcement of the bottom should consist of the same rods and the same spacing as the bin walls, and short rods should be imbedded in the upper face at the corners to take care of the negative moments.

The columns supporting the bin walls should be of liberal size, and the girder "a" formed by the intersection of the

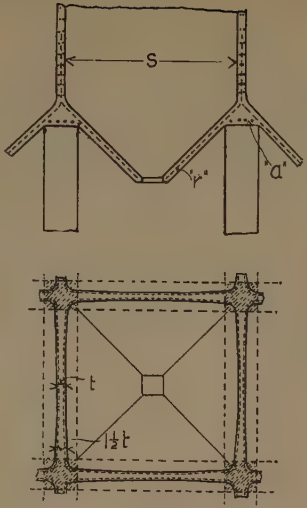


Fig. 59. Grain Elevators with Square Bins.

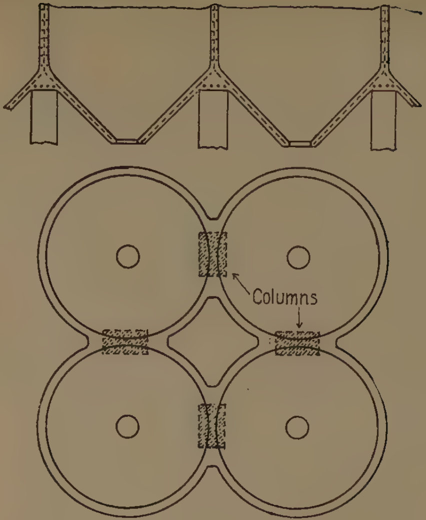


Fig. 60. Grain Elevators with Round Bins.

bottoms should have liberal dimensions and reinforced by at least four 1″ bars, in order to transmit the load from bin walls to columns.

In case of circular bins of a diameter d, the sectional area of the reinforcement for all points below 1.8d per lineal foot $= \frac{27.75d}{16000} \text{x} \frac{d}{2} = .000865d^2$, and can be diminished to ¼% at the top.

A suspended bottom is rarely used for bins more than 20′ in diameter; for larger diameters the bottom is supported by a regular floor construction. The shear per lineal foot equals again weight on bottom divided by circumference and produces a tension under 45° to be taken up by the suspension rods, and a compression at the bin wall of the values given in the eighth column. The horizontal reinforcement in the conical bottom should be the same as in the bin walls.

Also these bins should be supported by large columns and a heavy stiffening ring at the junction of bottoms and sides. In a cluster of circular bins, there are formed smaller bins, nearly square in section, which should be reinforced as given for square bins.

REINFORCED CONCRETE GRAIN ELEVATORS WITH SQUARE BINS.

Side of Square Bin Feet S	Horizontal Pressure 1.8 S Below Top	Thickness of Bin Wall in Center Inches	Reinforcement per Lineal Foot Sectional Area	Bushels per Lineal Foot.	Concrete per Lineal Foot Cubic Feet	Steel per Lineal Foot Pounds	Form Lumber per Lineal Foot Feet Board Measure	DATA FOR BOTTOM					Bushels in Bottom
								Vertical Shear per Lineal Foot of Circumference*	Stress per Lineal Foot Under 45°	Concrete Cubic Feet	Steel Pounds	Form Lumber Ft. B. M.	
6	168	4″	.120	45	10.7	17.3	48.0	450	638	24.2	72	150	91
8	222	4	.120	80	14.3	23.0	64.0	800	1130	42.5	130	250	215
10	278	5	.150	125	22.2	36.0	80.0	1250	1770	75.0	220	400	420
12	333	6	.250	180	32.0	65.0	96.0	1800	2550	126.0	376	570	725
13	360	7	.230	211	40.5	92.	104.0	2110	2980	166.0	600	670	925
14	389	7½	.31	245	46.6	120.	112.0	2450	3460	201.0	780	800	1170
16	440	8	.55	320	57.0	205.	128.0	3200	4530	272.0	1200	1000	1700

The vertical reinforcement=¼%.

One bushel=1.25 cubic feet.

*This shear=the radial force at the junction of bottom and side, producing bending stresses and the tensile forces in the bottom per lineal foot given in the tenth column.

DATA ON ROUND GRAIN OR COAL BINS.

Diameter D in Feet	Pressure per Square Foot at 1.8 D below Top	Section of Circular Reinforcement per Lineal Foot	Thickness of Shell Inches	One Lineal Foot of Cylinder contains Cubic Feet	Quantities per lineal foot			Data for Bottom				
					Concrete cubic feet	Steel pounds	Feet B. M.	Ring Stress at Joint Compression Lbs.	Stress per lineal foot in Bottom in the direction of 45°	Total Concrete cubic feet	Total Steel Lbs.	Forms Ft. B. M.
10	278	.120	4	78.5	10.8	31.2	125	6250	1760	37	113	560
12	333	.124	4	113.0	12.9	38.0	150	10800	2540	53.5	180	810
14	389	.171	4	153.5	15.0	53.0	172	17200	3450	72.2	341	1100
16	444	.222	4	200.0	17.0	71.5	196	25500	4500	95.0	873	1425
18	500	.282	5	253.0	24.2	101	220	36500	5700	147.0	917	1800
20	555	.346	5	314.0	26.6	129	245	50000	7040	182.0	1382	2225
22	611	.422	5	379.0	29.4	164	270					
24	667	.500	6	452.0	38.5	212	295					
26	722	.590	6	529.0	41.6	260	320					
28	778	.685	6	615.0	44.8	315	345					
30	834	.780	6	706	47.9	374	370					

Coal produces about the same pressure as grain.

Where the height of the bins is less than 1.8 diameters, the weight of the reinforcements may be reduced 20 to 40%.

CONCRETE DAMS.

Figures 61-62 show types of solid concrete dams, as generally executed in rubble masonry. In a dam, built according to Figure 61, the compressive stresses at the downstream edges of the base=100.5 h pounds, and those at the upstream edge= 68.5 h, while the greatest compressive stresses occur when the reservoir is empty, namely 166.5 h at the upstream edge. Such a dam is safe even if the water pressure should find its way under the base of the dam, a case which can only happen to any considerable extent if the dam is not founded on bedrock, an absolute necessity for any dam of importance.

In a dam built according to Figure 62, the compressive stresses at the downstream edge of the base=131 h, and those at the upstream edge=40 h; by giving the upstream face a batter of 8% the greatest compressive stresses, when the reservoir is full and when it is empty, are nearly alike, namely 150 h.

The rubble masonry consists generally of 50 to 60% of large, stones and 40 to 50% of concrete, 1:7½ to 1:9.

ARCHED DAMS.

The line of pressure for an arched dam is a portion of a circle. Let D=the length between the abutments in feet, and the rise of the arch=0.2 D, then the radius of the circle= 0.725 D and the stress per lineal foot height at a point h′ below the water line$=62.5\times h\times 0.725$ D$=45.3$ Dh; allowing a stress of 25000 pounds per square foot on rubble masonry, the thickness of the dam at a point h′ below the water line=0.00182 Dh. This would mean a feather edge for the top of the dam. If we assume the thickness at the top=0.1 of the base, the total volume per lineal foot of the section (the height of the dam =H′)=0.001 DH^2 cubic feet, or in the entire dam, considering that the length of the arch=1.10 D, are 0.0011 D^2H^2 cubic feet.

If the rise of the arch=0.288 D, the corresponding radius of the circle=0.58 D, and the thickness at the base=0.00145 DH, and the volume per lineal foot of the section=$0.000796\ DH^2$ cubic feet and the total volume of the dam, considering that the length of arch=1.209 D,=$0.000965\ D^2H^2$ cubic feet, which is the minimum volume obtainable in any arched dam.

Comparing these volumes with the volumes of the gravity dam, as shown in Figure 62, we find that the arched dam with a rise=0.2 D contains less material as long as D is less than 363′, and the dams with a rise=0.288 D contain less material as long as D is less than 415′.

It is an undisputed axiom that there is no stress where there is no deformation. If, on account of the friction, the base of the dam cannot move in accord with the proper elastic diminution of the length of the arch, there cannot be any arch action at the base, and the influence of such an immobility extends well up to the middle of the height. Hence, where there is no special provision made to allow of the safe sliding of the base, every arched dam must act as a gravity dam as well, because there is no other way for the structure to take care of that part of the water pressure which is not taken up by the arch action. It can be shown that in an arched dam, of a section as shown in Figure 63, the overturning moment about the base of that part of the water pressure which is not taken up by the arch action=$62.5\ \frac{h^3}{14}$, resulting in a compressive stress at the downstream edge of the base=177.5 H and in a compressive stress at the upstream edge of the base=9.5 H. The writer does not see any reason why the allowable stresses in dams should be limited to only 100 or 150 pounds per square inch, and he advocates the use of the same stresses which would be used in retaining walls. We see that arched dams of a section, as shown in Figure 63, show a saving of 38% in material over gravity dams of the section shown in Figure 62.

A far greater saving can be effected by the use of reinforced concrete arched dams, but inasmuch as such reinforced arched dams have not yet been built, we will not give any tables for these kind of dams.

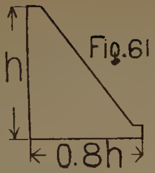


Fig. 61

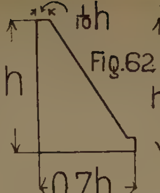


Fig. 62

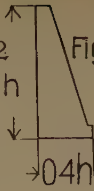


Fig. 63

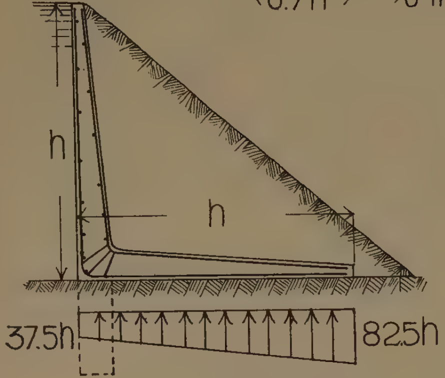


Fig. 64. Reinforced Concrete Dome, Cantilever Type.

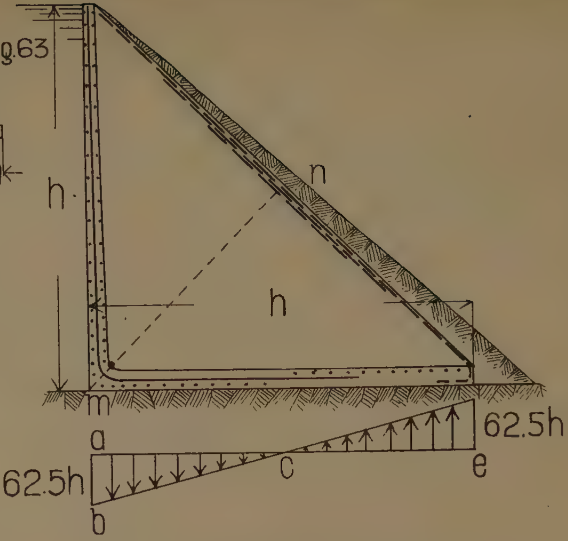


Fig. 65. Reinforced Concrete Dome with Counterforts.

REINFORCED CONCRETE DAMS.

The table on page 140 gives the data for cantilever type of reinforced concrete dams of moderate heights. A section of such a dam is shown in Figure 64. The stability of the dam is obtained by an earth fill on the downstream side. When the reservoir is empty the dam is strained, similar to a retaining wall of the type "A," and must be reinforced for earth pressure, and when the reservoir is full the dam is strained by the water pressure, which is partially counteracted by the earth pressure. On account of the triangular shape of the earth prism the earth pressure on the downstream face of the wall may be taken as $10h^2$, or the pressure per square foot at a point h below the top of the fill$=20\,h$, and the moment about the base of the wall$=3.33\,h^3$.

The moment from the water pressure about the base of the wall$=62.5\ \frac{h^2}{2} \times \frac{h}{3} = 10.42\,h^3$, or the back of the wall must be reinforced for a bending moment$=7.09\,h^3$.

The compressive stress at the downward edge of the base of the dam$=82.5\,h$ pounds per square foot and that at the upstream edge$=37.5\,h$, while the actual pressures, which must be taken care of by the base plate are given in Figure 65 by the diagram abcde.

The area of the reinforcing rods given in table on page 140 is for the intersection of wall and base, and the rods may be stopped off at proper intervals, according to the bending moments, which are acting at the various points.

The weight of the base and fill per square foot at the extreme downstream edge of the base must$=20\,h$ pounds, in order to make the structure safe against overturning when the reservoir is empty.

The yardage for the fill per lineal foot of the dam is given in the last column of the tables.

The very low stresses produced on the ground by these dams does not always necessitate the foundation of the dam on bedrock, provided an apron, as shown in dotted lines in Figure 64, is built at the upstream side of the dam into the impermeable strata below or unto bedrock. Where the building of this apron in the ordinary way would be too expensive, an apron of reinforced concrete sheet piles can be provided, and this apron can be made watertight by forcing grout into channels provided for this purpose in the sheet piles.

For dams higher than 25′ counterforts should be adopted, as shown in Figure 65. The data for these dams are given on page 141. The downstream face of the wall is figured for a pressure of 42.5 h pounds per square foot, and the upstream face for 20 h, and in both cases the bending moment was taken as $\frac{pl^2}{12}$, when p=pressure per square foot and l=clear span between counterforts. The base plate is stressed, as shown in Figure 65, by a distribution of loads given by diagram abcde, at the extreme edges by 62.5 h pounds per square foot, and a reversal of stresses takes place at the center of the base.

The counterforts are mostly stressed in the plane, marked mn in Figure 65, and the main reinforcement may be stopped off at proper interval. The reinforcement of the counterforts at the upstream side is extremely heavy for the higher dams and must be placed in the part of the wall adjoining the counterfort, and proper stirrups in horizontal direction must be provided to transmit the shear.

In case that there is danger that the dam is overtopped, the earth fill should be covered by a reinforced concrete slab at least 6″ thick, reinforced by ¼% in both directions.

REINFORCED CONCRETE DAMS.

(Cantilever Type.)

Height of Dam from the Base to Water Line	Thickness of Wall		Area of Vertical Reinforcement per Lineal Foot of Wall		UNIT QUANTITIES PER LINEAL FOOT			
	At the Base Inches	At the Top Inches	Down Stream Face	Up Stream Face	Cubic Feet of Concrete	Steel Pounds	Form Lumber Feet Board Measure	Earth Fill Cubic Yards
6	8″	8	.24	.24	8.0	28	24	.67
7	8″	8	.24	.24	9.5	34	28	.92
8	8″	8	.29	.24	10.5	40	32	1.40
9	10″	8	.30	.30	13.5	50	36	1.70
10	11″	8	.33	.33	16.0	61	40	2.10
11	12″	8	.50	.36	18.5	75	44	2.70
12	14″	8	.50	.42	22.0	85	48	3.20
13	15″	8	.63	.45	25.0	95	52	3.70
14	18″	8	.60	.54	30.0	108	56	4.20
15	18″	8	.90	.54	33.0	138	60	4.80
20	24″	8	1.44	.72	54.0	240	80	9.00
25	36″	8	1.50	1.08	92.0	350	100	13.60
30	48″	8	2.88	1.44	140.0	500	120	19.50

The longitudinal reinforcement = ½% at top, and ⅛ to ¼% at the lowest portion of wall.

REINFORCED CONCRETE DAMS WITH COUNTERFORTS.

Height of Dam from foundation to Water Line	Thickness of Wall Inches		Sectional area per lineal foot of horizontal Reinforcement in Wall				Counterforts				Base		Total Quantities per lineal foot			Earth Fill Cubic Yards
			At the Base		At the Top		Distance C. C.	Thickness	Sectional area of main Reinforcement		Thickness Inches	Area of Reinf't per lin.ft. at extreme up and down Stream Edges	Concrete Cubic ft.	Steel Lbs.	Form Lumber Ft. B. M.	
	at the Base	at the Top	Up Stream	Down Stream	Up Stream	Down Stream			Up Stream	Down Stream						
20	12″	8″	.36	1.2	.24	.24	15′–0″	1′–0″	3.14	1.76	20	.97	63	230	140	9
30	16	8	.48	1.5	.24	.24	15 –3	1 –3	3.55	2.00	20	1.32	115	410	260	19
40	18	10	.54	1.45	.30	.30	15 –6	1 –6	6.00	3.14	24	1.44	201	670	410	34
50	22	12	.66	1.32	.36	.36	16 –0	2 –0	10.00	5.00	27	1.62	334	900	580	50
60	24	12	.72	1.44	.36	.36	16 –0	2 –0	12.5	6.25	30	1.80	453	1200	770	69
70	26	12	.78	1.56	.36	.36	16 –3	2 –3	14.6	7.30	32	1.92	611	1500	960	93
80	30	12	.90	1.80	.36	.36	17 –6	2 –6	23.5	11.70	36	2.16	825	2160	1180	120
90	32	12	.96	1.92	.36	.36	17 –6	2 –6	31.5	15.70	39	2.34	1008	2760	1500	150
100	38	18	1.14	2.28	.54	.54	20 –0	3 –0	40.0	20.00	46	2.76	1330	3450	1600	182
125	42	18	1.26	2.52	.54	.54	20 –6	3 –6	57.00	27.50	52	3.12	2165	5310	2400	280
150	47	18	1.42	2.82	.54	.54	21 –0	4 –0	77.00	38.50	57	3.42	3200	7470	3100	400
175	50	18	1.50	3.00	.54	.54	22 –0	5 –0	92.00	46.00	60	3.60	4780	9900	4100	545
200	54	18	1.62	3.24	.54	.54	23 –0	6 –0	154.00	77.00	66	3.96	6800	14800	5000	580
250	60	18	1.80	3.60	.54	.54	24 –0	7 –0	204.00	102.00	75	4.50	11200	23200	7200	1020

REINFORCED CONCRETE WATER PIPES.

Let d=inside diameter in inches, and p=pressure in pounds per square inch (a head of one foot of water means a pressure of 0.43 pounds inch^2, or a pressure of 1 pound inch^2 means a head of 2.31 feet), 12000=safe stress per square inch on the helical reinforcement, then the sectional area of the helical reinforcement per lineal foot= $\frac{dp}{2000}$. Where occasional loss of water is of no great account a stress of 16000 pounds per square inch may be allowed.

The pipes are generally made in lengths of 3′ to 10′, in permanent or temporary shops, always by the use of sheet-steel forms. Their connections are formed by means of sleeves, generally 8″ long and about 1″ larger in diameter than the pipes, and the space between sleeves and pipes is filled with grout.

For heads between 80′ to 100′, a mixture of not less than one part cement to 1.3 parts of sand and fine gravel must be used. For heads between 50′ to 70′ the mixture should not be less than 1:1.5, for heads between 30′ to 40′ not less than 1:2, for smaller pressures not less than 1:2.25.

The pitch of the reinforcement should be very small, from 1½″ for 100′ head down to 4″ for small heads.

Where the friction on the ground takes up the greatest portion of the tension in the pipes, the longitudinal reinforcement may be limited to ¼%, otherwise the sectional area of the longitudinal reinforcement per lineal foot of circumference must be one-half of the sectional area of the helical reinforcement per lineal foot.

For higher pressures than 45 pounds inch^2 the water tightness is obtained by a sheet-steel lining $\frac{1}{32}$″ to ⅛″ thick, which in important pipes is protected by an inside reinforced concrete shell 1″ thick.

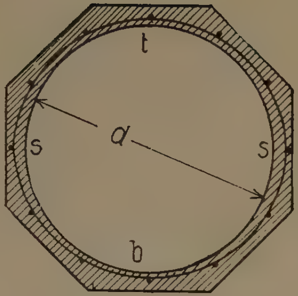


Fig. 66. Cross Section of Sewer.

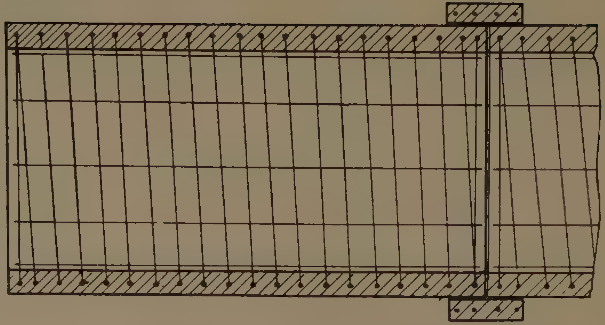


Fig. 67. Longitudinal Section of Water Pipe with Sleeve Connection.

PROPERTIES OF WATER PIPES.

	10"					12"					15"				
Pressure Pounds per Square Inch	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot
10	1"	.050	.75	35	.25	1"	.06	1.00	41	.29	1¼"	.075	1.55	64	.45
15	1"	.075	.97	35	.25	1"	.09	1.32	41	.29	1¼"	.1125	1.98	64	.45
20	1"	.100	1.20	35	.25	1¼"	.12	1.73	53	.37	1¼"	.15	2.59	64	.45
25	1"	.125	1.43	35	.25	1¼"	.15	2.06	53	.37	1½"	.187	3.20	78	.54
30	1"	.150	1.66	35	.25	1¼"	.18	2.40	53	.37	1½"	.225	3.73	78	.54
35	1"	.175	1.89	35	.25	1¼"	.21	2.72	53	.37	1½"	.262	4.25	78	.54
40	1¼"	.200	2.19	45	.32	1½"	.24	3.13	64	.45	1½"	.300	4.76	78	.54
45	1¼"	.225	2.41	45	.32	1½"	.27	3.46	64	.45	1½"	.337	5.28	78	.54
	18"					20"					24"				
10	1¼"	.09	2.10	76	.53	1½"	.10	2.67	101	.71	1½"	.12	3.63	120	.84
15	1¼"	.135	2.85	76	.53	1½"	.15	3.60	101	.71	1½"	.18	4.95	120	.84
20	1½"	.180	3.71	92	.64	1½"	.20	4.50	101	.71	1½"	.24	6.28	120	.84
25	1½"	.225	4.45	92	.64	1½"	.25	5.58	101	.71	1½"	.30	7.60	120	.84
30	1½"	.270	5.20	92	.64	1¾"	.30	6.50	119	.83	1¾"	.36	9.08	142	1.00
35	1¾"	.315	6.05	109	.76	1¾"	.35	7.40	119	.83	1¾"	.42	10.40	142	1.00
40	1¾"	.360	6.80	109	.76	2"	.40	8.45	138	.96	2"	.48	11.90	164	1.14
45	1¾"	.405	7.53	109	.76	2"	.45	9.40	138	.96	2"	.54	13.20	164	1.14

The longitudinal reinforcement is assumed = ¼%.

PROPERTIES OF WATER PIPES.

Inside Diameter	27"					30"					36"				
Pressure Pounds per Square Inch	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot
10	1½"	.135	4.45	134	.94	1½"	.15	5.37	149	1.04	1¾"	.18	7.70	208	1.45
15	1½"	.202	6.10	134	.94	1½"	.225	7.42	149	1.04	1¾"	.27	10.60	208	1.45
20	1½"	.270	7.80	134	.94	1¾"	.300	9.70	175	1.22	1¾"	.36	13.60	208	1.45
25	1¾"	.337	9.65	158	1.10	1¾"	.375	11.80	175	1.22	1¾"	.45	16.60	208	1.45
30	1¾"	.405	11.30	158	1.10	1¾"	.450	13.80	175	1.22	2"	.54	19.80	239	1.66
35	1¾"	.472	13.00	158	1.10	2"	.525	16.10	202	1.40	2"	.63	22.80	239	1.66
40	2"	.540	14.80	182	1.34	2"	.600	18.10	202	1.40	2"	.72	25.90	239	1.66
45	2"	.607	16.50	182	1.34	2"	.675	20.30	202	1.40	2"	.81	28.90	239	1.66
Inside Diameter	42"					48"					54"				
10	1¾"	.210	10.10	241	1.68	2"	.24	13.20	315	2.18	2¼"	.27	16.7	398	2.76
15	1¾"	.315	14.20	241	1.68	2"	.36	18.50	315	2.18	2¼"	.405	23.40	398	2.76
20	1¾"	.420	18.20	241	1.68	2"	.48	23.80	315	2.18	2¼"	.540	30.00	398	2.76
25	1¾"	.525	22.20	241	1.68	2¼"	.60	29.40	356	2.48	2½"	.675	37.10	444	3.10
30	2"	.630	26.60	277	1.93	2¼"	.72	34.60	356	2.48	2½"	.810	43.80	444	3.10
35	2"	.735	30.60	277	1.93	2½"	.84	42.00	398	2.76	2¾"	.945	51.00	490	3.40
40	2¼"	.840	35.00	313	2.18	2½"	.96	45.50	398	2.76	2¾"	1.08	57.80	490	3.40
45	2½"	.945	39.30	350	2.43	2½"	1.08	50.80	398	2.76	2¾"	1.22	64.50	490	3.40

The longitudinal reinforcement is assumed=¼% and included in the weight of steel.

PROPERTIES OF WATER PIPES.

Inside Diameter	60″					66″					72″				
Pressure Pounds per Square Inch	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot
10	2½″	.30	20.6	491	3.42	2½″	.33	24.50	539	3.74	3″	.36	29.7	708	4.91
15	2½″	.45	28.8	491	3.42	2½″	.495	34.50	539	3.74	3″	.54	41.50	708	4.91
20	2½″	.60	37.00	491	3.42	2½″	.66	44.50	539	3.74	3″	.72	53.3	708	4.91
25	2½″	.75	45.30	491	3.42	2½″	.825	54.40	539	3.74	3″	.90	65.2	708	4.91
30	3″	.90	54.30	594	4.13	3″	.99	65.40	651	4.52	3½″	1.08	78.10	851	5.91
35	3″	1.05	62.80	594	4.13	3″	1.15	75.00	651	4.52	3½″	1.26	90.00	851	5.91
40	3½″	1.20	72.00	698	4.85	3½″	1.32	86.10	765	5.30	4″	1.44	103.0	956	6.65
45	4″	1.35	81.00	805	5.58	4″	1.48	97.00	880	6.11	4″	1.62	115.00	956	6.65
Inside Diameter	78″					84″					90″				
10	3½″	.39	35.40	897	6.22	4″	.42	41.50	1107	7.67	4″	.45	47.00	1182	8.20
15	3½″	.585	49.30	897	6.22	4″	.63	57.80	1107	7.67	4″	.675	65.60	1182	8.20
20	4″	.78	64.30	1035	7.16	4″	.84	74.00	1107	7.67	4″	.90	84.30	1182	8.20
25	4″	.975	78.50	1035	7.16	4½″	1.05	91.50	1252	8.70	4½″	1.125	104.00	1337	9.28
30	4½″	1.17	93.50	1167	8.10	5″	1.26	109.00	1399	9.70	5″	1.35	124.00	1493	10.80
35	5″	1.365	108.00	1304	9.05	5″	1.47	125.00	1399	9.70	5″	1.575	142.50	1493	10.80
40	5″	1.56	122.50	1304	9.05	6″	1.68	143.00	1697	11.80	6″	1.80	164.00	1810	12.60
45	6″	1.755	139.00	1584	1.10	6″	1.89	160.00	1697	11.80	6″	2.025	182.00	1810	12.60

The longitudinal reinforcement is assumed=¼% and included in the weight of steel.

PROPERTIES OF WATER PIPES.

Inside Diameter	96"					102"					108"				
Pressure Pounds per Square Inch	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot	Thickness of Shell in Inches	Area of Helical Reinforce't per Lin. Foot	Weight of Steel per Lineal Foot	Weight of Pipe per Lineal Foot	Cubic Feet of Concrete per Lineal Foot
10	5"	.48	55.50	1587	11.1	5"	.51	61.90	1681	11.7	6"	.54	71.10	2150	15.0
15	5"	.72	76.50	1587	11.1	5"	.765	85.50	1681	11.7	6"	.81	97.90	2150	15.0
20	6"	.96	100.50	1823	12.6	6"	1.02	112.00	2036	14.2	7"	1.08	128.00	2530	17.6
25	6"	1.20	122.00	1823	12.6	6"	1.275	136.00	2036	14.2	7"	1.35	154.50	2530	17.6
30	7"	1.44	145.50	2266	15.8	7"	1.53	163.00	2398	16.60	7"	1.62	181.00	2530	17.6
35	7"	1.68	167.00	2266	15.8	7"	1.78	186.00	2398	16.60	8"	1.89	211.00	2916	20.3
40	8"	1.92	190.00	2614	18.2	8"	2.04	214.00	2765	19.3	8"	2.16	237.00	2916	20.3
45	8"	2.16	211.00	2614	18.2	8"	2.29	237.00	2765	19.3	8"	2.43	265.00	2916	20.3
Inside Diameter	114"					120"					132"				
10	6"	.57	78.10	2262	15.8	6"	.60	85.90	2376	16.5	6"	.66	102.00	2602	18.1
15	6"	.855	108.00	2262	15.8	6"	.90	119.00	2376	16.5	6"	.99	142.00	2602	18.1
20	7"	1.14	141.00	2661	18.5	7"	1.20	155.00	2794	19.5	7"	1.32	185.00	3058	21.2
25	7"	1.425	171.00	2661	18.5	7"	1.50	188.00	2794	19.5	7"	1.65	225.00	3058	21.2
30	7"	1.71	200.00	2661	18.5	7"	1.80	221.00	2794	19.5	7"	1.98	265.00	3058	21.2
35	8"	1.995	234.00	3067	21.3	8"	2.10	257.00	3218	22.3	8"	2.31	309.00	3520	24.5
40	8"	2.28	263.00	3067	21.3	8"	2.40	290.00	3218	22.3	8"	2.64	350.00	3520	24.5

The longitudinal reinforcement is assumed=¼% and included in the weight of steel.

REINFORCED CONCRETE CIRCULAR SEWERS.

We know very little of the forces acting on a sewer in the ground. The vertical loads produce a positive moment at the top "t" and negative moments at the sides "s," but we are unable to tell in what manner the earth pressure counteracts these moments. It is also clear that there is a certain arch action in the fill above the sewer (similar to what was said in chapter on Grain Elevators), preventing the full weight of the fill acting on the sewer, especially in deep trenches; also that the live head is only partially transmitted to the sewers. In many German cities the sewers for heavy traffic are designed for the following vertical dead and live loads for various depth of fill, neglecting the favorable influence of the earth pressure.

Height of fill	3′	7′	15′
Vertical load per square foot....	800 lbs.	700 lbs.	600 lbs.

By the theory of least work, we obtain the following moments for a uniform, vertical load p per square foot acting over the entire diameter, over ½ diameter and for a concentrated load P in center.

LOAD	pd	$\frac{pd}{2}$	P
Positive Moment at t & b................................	$\frac{pd^2}{16}$	$\frac{pd^2}{20}$	$\frac{Pd}{6.28}$
Negative Moment at S & S	$\frac{pd^2}{16}$	$\frac{pd^2}{23.5}$	$\frac{Pd}{11.1}$

when d=diameter in feet.

The sewers on page 149 were figured for a moment of $\frac{pd^2}{16}$ and it is clear that the reinforcement must be placed at top and bottom near the inside face and in the center near the outside face. Where sewers are built in the ground it is as a rule cheaper to give the outside an octagonal shape, as shown in Figure 66. The longitudinal reinforcement may be limited to ⅛%.

CIRCULAR SEWERS.

	Load per Square Foot Lbs.	DIAMETER OF CIRCULAR SEWER						
		2'	3'	4'	5'	6'	7'	8'
Maximum Bending Moment..	800	200	450	800	1250	1800	2450	3200
	700	175	395	700	1100	1580	2150	2800
	600	150	333	600	940	1350	1840	2400
Thickness of Ring and Sectional Area of Reinforcement per lineal foot......	800	2"	3"	4"	5"	6"	7"	8
		.06	.09	.120	.150	.180	.210	.240
	700	1¾"	2½"	3½	4½	5½"	6"	7"
		.06"	.12	.17	.190	.165	.250	.252
	600	1½"	2¼	3½	4"	5"	5½	6½
		.06"	.09	.105	.190	.180	.240	.280

		DIAMETER OF CIRCULAR SEWER						
		9'	10'	11'	12'	13'	14'	15'
Maximum Bending Moment..	800	4050	5000	6050	7200	8450	9800	11250
	700	3550	4370	5300	6300	7400	8550	9800
	600	3050	3750	4550	5400	6350	7350	8400
Thickness of Ring and Sectional Area of Reinforcement per lineal foot......	800	9"	10"	11"	12"	13"	14"	15"
		.270	.320	.330	.36	.39	.42	.45
	700	8"	9"	10"	11"	12"	13"	14"
		.290	.270	.30	.36	.36	.39	.42
	600	7"	8"	9"	10"	11"	12"	13"
		.340	.330	.320	.30	.36	.36	.390

REINFORCED CONCRETE PILES.

The reinforcement of concrete piles is only governed by the requirements of safe handling before the actual driving. Experience also demonstrates that small cracks, occasioned by the transport of the piles, do not cause failure of the piles in driving; the fact is they close up after the first blow. The piles in the table are figured for a moment $\frac{pl^2}{20}$ when p the weight per lineal foot in pounds, and l the length of the pile in feet. This moment is produced when the pile is supported at one end and at a point 0.3 l away from the other end. This gives a factor of safety of 1.6 for the case, that by accident the pile is supported at both ends only. Unless the piles are driven by a water jet it is necessary to use very heavy hammers for driving, in order to reduce the cost of driving. It is desirable, in hard ground, to use a hammer about twice the weight of the pile and not much less than the weight of the pile; then the driving does not take more time than that of an ordinary wooden pile.

Let h=the fall of the hammer in inches,
e=average of the last ten blows in inches,
H=weight of hammer in pounds,
P=weight of pile in pounds;

then the weight which the pile can carry is given by the formula $W = \frac{h}{e} \frac{H^2P}{(H+P)^2}$ pounds. This formula nearly always gives satisfactory results.

h is rarely more than 90″.

Rarely less than 25 tons nor more than 80 tons are allowed on a concrete pile.

In filled ground it is of advantage to give the piles a taper, making the point not less than 6″. If the center of the piles is made of the thickness given in the tables, then the same reinforcement may be used, if the pile is handled 0.30 l away from the point. In good fill it is safe to figure with a frictional resistance of 1000 pounds per square foot.

For wharf work, where great stiffness is required, hollow piles of diameters up to 5′ are preferable to solid piles.

DATA ON REINFORCED CONCRETE PILES.

Length of Pile	Side of Square Pile Inches	Bending Moment in 1000 Foot Pounds	Theroetically Required Reinforcement 4 Rounds	Weight of Steel Pounds	Cubic Feet of Concrete in One Pile	Weight of Pile Pounds	Safe Load 1000 Pounds	Mixture
20′	12	2.9	$\frac{5}{8}$″	†138	20	2900	77	1:6
25′	12	4.5	$\frac{5}{8}$″	†173	25	3600	77	1:6
30′	12	6.5	$\frac{5}{8}$″	†207	30	4350	77	1:6
35′	14	12.3	$\frac{3}{4}$″	248	48	6900	100	1:6
40′	14	16.0	$\frac{7}{8}$″	368	54	7800	104	1:6
45′	14	20.3	1″	526	61	8800	109	1:6
50′	14	25.0	1$\frac{1}{8}$″	725	68	9800	115	1:6
55′	14	30.3	1$\frac{1}{4}$″	970	75	10800	121	1:5
60′	14	36.0	1$\frac{3}{8}$″	1270	82	11800	128	1:5
65′	15	47.5	1$\frac{1}{2}$″	1630	102	14700	149	1:4
70′	15	55.0	1$\frac{3}{4}$″	2360	110	15900	167	1:4
75′	16	72.0	1$\frac{7}{8}$″	2900	133	19200	191	1:3
80′	16	82.0	2″	3520	142	20500	201	1:3
85′	18	117.0	*1$\frac{7}{8}$″	4900	191	27500	256	1:3
90′	18	131.0	*2″	6000	202	29000	272	1:3
100′	20	200.0	‡2″	9700	278	40000	350	1:2

* 6 rods. ‡ 8 rods.

† Assumed 4-¾ rods.

The piles should be well cured, repeatedly drenched and then sprinkled with water, and should be two months old before driving. Piles which are driven by a hammer within one month or even three weeks, should contain at least 50% more cement than indicated above.

ARCHED BRIDGES

The most favorable shape of an arched bridge without hinges is a curve very closely following a parabola.

For the dead load and any fixed loading we can find by the theory of catenary curves the line of resistance which coincides with the center of gravity of each section of the arch, and, therefore, produces uniform pressure in each point of a section, but any other loading will then produce bending moments in all points of the arch. It can be shown that the most economical shape of an arch is very closely the line of pressure which corresponds to the full dead load and one-half of the live load, uniformly distributed. We have, as a rule, two types of arched bridges: one as shown in Figure 76, with solid spandril walls and dirt fill above the arch; and one as shown in Figure 77, with open spandril construction.

Let 100a=the weight of the arch ring per square foot at the apex+the weight of the dirt fill and pavement per square foot+50 pounds, which is one-half of the customary live load of 100 pounds per square foot, and f=the rise of the arch in feet, 100=weight of dirt fill per cubic foot, and l=span in feet then the line of pressure for the first type of arched bridges is very closely given by

$$y=\frac{8f}{1+6\frac{a}{f}}\left\{(1+3\frac{a}{f})\frac{x}{l}-3(1+\frac{a}{f})(\frac{x}{l})^2+4(\frac{x}{l})^3-2(\frac{x}{l})^4\right\}\ldots($$

when the origin of the system of coördinates is at one abutment and the axis of the abscissae is horizontal. The horizontal projection of the thrust is given by $T=100\frac{l^2}{48}(1+6\frac{a}{f})$ (2)

and $tgb=\frac{8f}{l}\frac{1+3\frac{a}{f}}{1+6\frac{a}{f}}$ (3), where b is the angle of the tangent of the line of pressure with the horizontal at the abutment.

In the following table we can find the ordinates for the various 1/10 points of the arch and for the different values of $\frac{a}{f}$, by multiplying the black figures by f.

$\frac{a}{f}$	$\frac{H}{l^2}$	**0.4 x**	**0.3 x**	**0.2 x**	**0.1 x**	tgb ÷ $\frac{f}{l}$
.5	8.33	.969	.873	.697	.412	5.00
.45	7.71	.970	.879	.703	.418	5.58
.40	7.09	.971	.880	.706	.426	5.18
.35	6.46	.972	.885	.715	.435	5.32
.30	5.83	.975	.887	.722	.442	5.41
.25	5.21	.976	.889	.733	.450	5.60
.20	4.58	.978	.901	.745	.464	5.81
.15	3.96	.985	.913	.762	.482	6.11
.10	3.33	.987	.924	.784	.505	6.50
Parabola....................		.96	.84	.64	.36	4.00

In the open spandril type of arched bridges the line of pressure varies from a parabola only on account of the increase of weight of the arch towards the abutment.

Let d=weight of arch per square foot at the apex+weight of pavement and floor construction at apex per square foot +50 pounds, and c=the increase of weight per square foot of the arch at the abutment, then the line of pressure is given by

$$y=\frac{8f}{1+\frac{3d}{c}}\left\{\left(\tfrac{3}{4}+\frac{3d}{2c}\right)\frac{x}{l}-\left(\tfrac{3}{2}+\frac{3d}{2c}\right)\left(\frac{x}{l}\right)^2+\left(\frac{x}{l}\right)^3\right\}\ldots.(4)$$

The horizontal projection of the thrust$=\frac{l^2}{8f}\left(d+\frac{c}{3}\right)\ldots(5)$

and $\text{tgb}=\frac{8\left(\frac{3}{4}+\frac{3d}{2c}\right)}{1+\frac{3d}{c}}\frac{f}{l}..(6)$ practically the same as the tangent to a parabola, Inasmuch as the shape of these curves is very near a parabola, we propose to find the bending moment, thrust and reaction at the abutment from a concentrated or moving uniform load by the theory of parabolic arches of constant ratio of ds: I.

In textbooks can be found the following or similar equations for the reaction (R) thrust (T) and moment (Ma) at the left abutment, for a concentrated load P, placed at a point u from the left abutment:

$$R = P\frac{(l-u)^2(l+2u)}{l^3},\quad T = P \times \frac{15}{4l^3f}\ \frac{(l-u)^2u^2}{(1+k)}$$

$$Ma = P\frac{u(l-u)^2}{l^3}\ (1-\frac{2.5u}{1+k})$$

when l=span in feet, f=rise in feet and $k = \frac{I}{Af^2} \times \frac{45}{4}$ wherein I=moment of inertia of the cross section, which is assumed to be constant, and A=area of the cross section. The influence of the shortening of the arch by the thrust is represented by k, but by inspecting the tables of arched bridges it will be found that k is never larger than 0.03 and mostly very much less, therefore we shall omit it in our calculations.

We can figure the values of R, T and Ma for the various positions of a moving concentrated load from above equations; this was done for each 1/10 point of the arch and shown graphically in Figures 69 to 71. The curves are called the influence line for R, T and Ma, respectively. We consider a moment positive if it tends to bend the arch in the same way as a downward load in a freely supported girder. Having found the indeterminate values of R, T and Ma, it is easy to find the moment about any point of the arch by the equations $M = Rx - Ty - Ma$ and $M = Rx - Ty - Ma - P(x-u)$, according to whether the moment is to be found for a point at the left or the right of P.

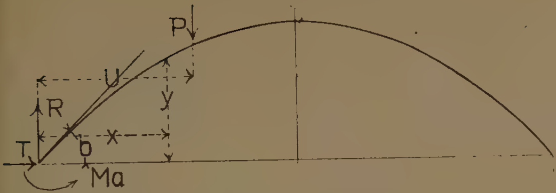


Fig. 68.

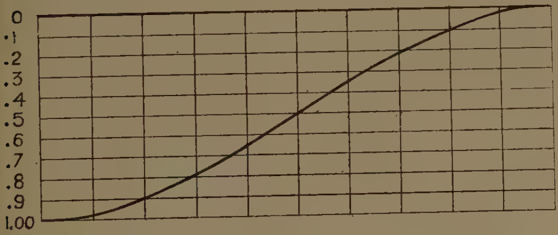


Fig. 69.

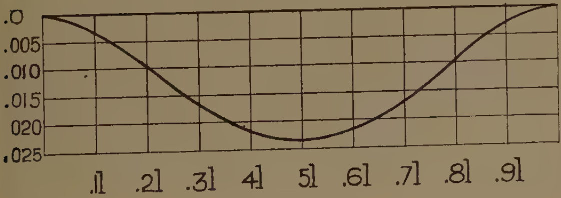


Fig. 70.

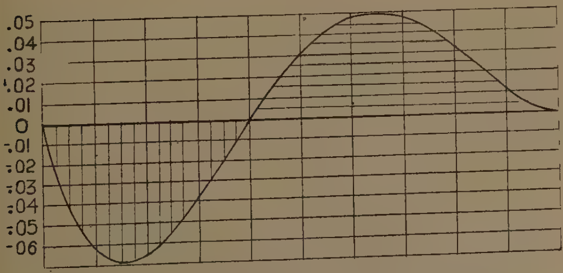


Fig. 71.

Influence Lines for Reaction, Thrust and Moment at Left Abutment in a Parabolic Arch.

The ordinates of the curve in Figure 72 give, if multiplied by P.l, the moments from a moving concentrated load P about a section of the arch at the 2/10 point, and is called the influence line of the moments for the point x=0.2 l. We see that the greatest positive moment is produced when the load is at the point x=0.2 l, that no moment is produced at the section x=0.2 l, when the load is at x=0.36 l, and that negative moments are produced when the load moves further to the right; and the greatest negative moment about the section x=0.2 l is produced when the load is at x=0.54 l.

The greatest positive moment from a moving uniform load about the section x=0.2 l will be produced, when the load moves from the left abutment to the point x=0.36 l and is found by multiplying area of the diagram below the axis of the abscissae by pl, when p=uniform load per square foot. The greatest negative moment is produced when the uniform load moves from the right abutment up to the point x=0.36 l.

By inspecting the Figures 72-75, it will be found that the greatest moment from a concentrated load about any point of the arch is produced at the section x=0.3 l, when the load is placed at the same section, and=0.0596 Pl, a very high value if we consider that the bending moment in a girder =0.25 Pl. The greatest negative moment about any point of the arch is also produced at the section x=0.3 l, when the load is at x=0.6 l.

The maximum bending moment from a moving uniform load is also produced at the section x=0.3 l, when the load moves from the left abutment up to the point x=0.44 l; at the same section is also produced the largest negative bending moment in any point of the arch, when the movable load moves from the right abutment to the point x=0.44 l.

The following table gives the maximum moments and the corresponding thrust from a concentrated load and from a movable uniform load for the sections of the arch at x=0.2, 0.3, 0.4, and 0.5 l:

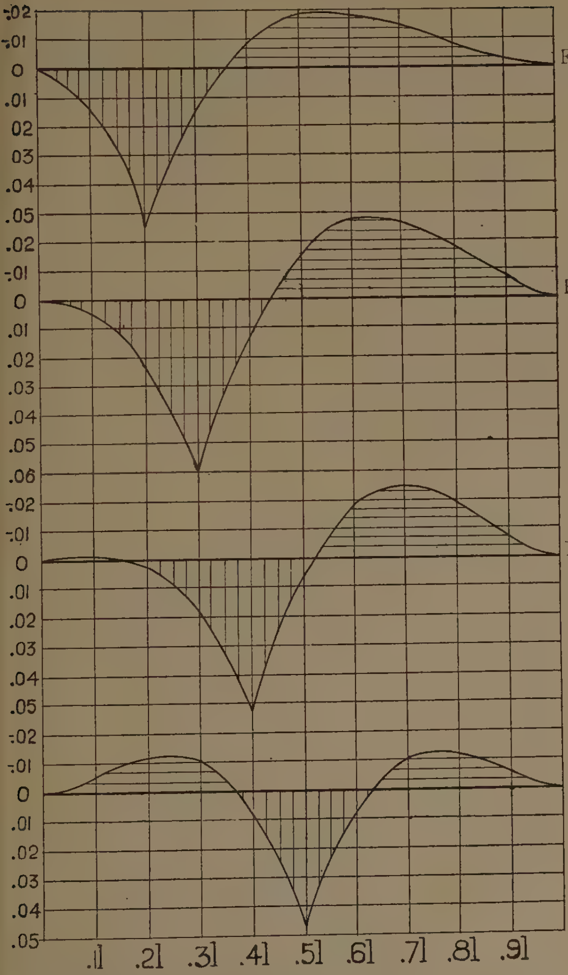


Fig. 72.

Fig. 73.

Fig. 74.

Fig. 75.

Influence Lines for Moments at x=0.2 l, 0.3 l, 0.4 l, and 0.5 l.

x =	0.2 l	0.3 l	0.4 l	0.5 l
Maximum positive moment from a moving uniform load pl^2 times	0.0069	0.0096	0.0066	0.0057
" Ma (left abutment) " " pl^2 "	0.0174	0.0169	0.0152	0.0078
" Thrust " " $p\frac{l^2}{f}$ "	0.0323	0.047	0.067	0.060
" Negative Moment " " pl^2 "	0.0067	0.010	0.0064	0.0053
" Ma (right abutment) " " pl^2 "	0.0096	0.0141	0.0162	0.0078
" Thrust " " $p\frac{l^2}{f}$ "	0.0925	0.077	0.058	0.065
Maximum positive Moment for a concentrate load Pl "	0.0542	0.0598	0.0522	0.0462
" negative " " " " Pl "	0.019	0.028	0.0247	0.0125

From the foregoing we can deduce the following rules for the design of an arched bridge:

After having decided on the type of bridge, we find the shape of the arch by equation No. 1 or No. 4, and the thrust from the dead load and one-half of the live load by equations No. 2 or No. 5. The greatest moment in the arch is produced at the section x=0.3 l and $=\frac{pl^2}{100}$ from a uniform load, or for a concentrated load can be found as Pl times the ordinates in Figure 73, when axle loads are so grouped around the apex of the influence line so that the summation of the ordinates times the various axle loads is a maximum. We assume that the load from a wagon is distributed by the fill and transverse stiffness of the arch over 12′ of the width of the arch, hence, the values for the bending moment from a wagon load must be divided by 12 to obtain the moment for one foot width of the arch.

In the tables on pages 170 to 177 are given the values of a:f, which we assumed sufficiently high, of the thrust per lineal foot width of the bridge from dead and half the live load, and the moment from a uniform load, moving from the left abutment to x=0.43 l, about the section of the arch at x=0.3 l, and also the moments per lineal foot width of the arch from a 12 and 24 ton wagon (Cooper's specifications), and the corresponding thrusts, also for the section at x=0.3 l.

Up to and including the spans of 140′, the type with solid fill was adopted; for the spans 150′ to 180′ open spandril construction with solid arch; for larger spans open spandril construction with ribbed arches, connected by a concrete slab.

For three different rises of the arch for each span is given the required thickness of the arch at the crown, and this thickness should be gradually increased by 50% at the abutment. In all solid arches we assumed that both the top and bottom of the arch ring in reinforced by ½% of steel in the direction of the bridge and by ⅛% cross ways (both top and bottom).

The use of the tables is best explained by an example: We shall assume that a bridge of a span of 120′ and a rise of 20′ is to be designed. From the table on page 173 we find that the greatest moment from a uniform load at the 3/10 section is 14400 foot-pounds, and from a 24-ton wagon 26000 foot-pounds; hence, the uniform load is more unfavorable than a 12-ton wagon, which would only produce a moment of 13000 foot-pounds.

We do not make any appreciable error if we assume that the thrust given under the heading T, which includes the thrust from the dead load and half the uniform live load, is the thrust for a uniform load which proceeds from the right or left abutment up to $x=0.44\ l$; hence, the arch ring must be figured for a thrust=65000 pounds and a bending moment of 14400 foot-pounds at the point $x=0.3\ l$ for a moving uniform load.

On page 167 we find that a 18″ slab, reinforced by ½% at top and at the bottom, has an area of 248 square inches and a moment of resistance of 865 inch^3, and the compressive stresses from the thrust$=65000\div248=262$ pounds per square inch, and the greatest compressive and tensile stresses at the extreme fibres from the bending moment $=12\times14400\div865=200$ pounds per square inch, or the greatest compressive stress$=262+200$ pounds$=462$ pounds per square inch.

We advocate an allowable stress of 600 pounds per square inch, hence the design is perfectly safe.

In order to take care of the few imperfections in our assumptions we shall take the section figured for the 3/10 point of the arch as the crown section, and shall increase the thickness of the arch (but not the reinforcement) to 1½ times the thickness of the crown at the abutment, which means, of course, also an increased thickness at the 3/10 point. Inasmuch as the positive and negative moments are practically alike, it is clear that the reinforcement must be the same at both the top and the bottom of the arch. The greatest absolute values of the thrust and the moment are at the abutment, and the thrust is 58% greater than 65000, and the moment is 69% greater than 14400; as a rule it will be found that by making the thickness of the arch 1½ times that of the crown that the stresses of the abutment are less than those for the 3/10 point.

If the bridge is to be figured for a 24-ton wagon, we obtain the thrust by substracting from the value given under T one-half of the thrust from the uniform load given under the heading 100 $\frac{l^2}{8f}$, and adding the thrust given under the heading T from a 24-ton wagon, which in this case=64000 pounds per lineal foot width of the bridge. The moment from a 24-ton wagon load is given as 26000 foot-pounds, and assuming an arch ring 20″ thick, of an area of 276 square inches and a moment of resistance of 1075 inch^3, we find the compressive stresses from the thrust=64000 : 276=233 pounds per square inch, and the stress from the bending moment =12×26000 : 1075=291 pounds per square inch, or a total stress of 524 pounds.

The stresses from a variation of temperature are found by the following consideration:

Let E=modul of elasticity=2,000,000 lbs. per square inch,
c=coefficient of expansion for 1° F=0.0000055,
t=change of temperature=50° F,
I moment of inertia in inch^4
f=rise of arch in inches;

then by the theory of least work and for a constant relation of ds:I in a parabolic arch, the thrust from a change of temperature= $\frac{45\,E\,I\,ct}{4f^2}$ pounds, and the moment at the abutment$=\frac{15\ \mathrm{EI\,ct}}{2\mathrm{f}}$ inch-pounds; substituting the above values for E, c and t, we have $\mathrm{T}=6200\frac{\mathrm{I}}{\mathrm{f}^2}$ lbs and $\mathrm{Ma}=4140\frac{\mathrm{I}}{\mathrm{f}}$ inch pounds. The moments at the various 1/10 points of the arch are for x= 0.2, 0.3, 0.4, 0.5 1

$$-170\frac{\mathrm{I}}{\mathrm{f}}\quad 940\frac{\mathrm{I}}{\mathrm{f}}\quad 1790\frac{\mathrm{I}}{\mathrm{f}}\quad 2060\frac{\mathrm{I}}{\mathrm{f}}\ \text{inch lbs.}$$

In above example the moment of inertia for a width of one foot is given on page 167=7782, the rise =240″, therefore the bending moment from a variation of temperature at the 3/10 point $940\frac{\mathrm{I}}{\mathrm{f}}=940\times\frac{7782}{240}=30500$ inch-pounds, and the corresponding stress in the extreme fibres=30500÷865=36 pounds per square inch. In the tables on pages 180-183 are all these stresses tabulated. The increase of stress from the thrust by the change of temperature is very small and may be neglected. Also the stresses due to the shortening of the arch are very small. The bending moment at the crown of the arch is very close$=\frac{\mathrm{pl}^2}{24}\ \frac{\mathrm{k}}{1+\mathrm{k}}$, when p the average total dead and live load per square foot. We said that k is never larger than 0.03 in the bridges given in our tables; therefore the bending moment due to the shortening of the arch is small in comparison to the moments from the live loads. It is good practice to limit the extreme fibre stresses in the arch, including all temperature stresses, to 700 pounds per square inch.

It may be objected that we have no constant ds:I in our arch. By a more exact calculation can be found that an increase of the thickness towards the abutment increases the stresses at the abutments and decreases the stresses in the arch proper, and this variation amounts only to two to four per cent in the case that the thickness at the abutment=1½ thickness at the crown.

We have seen, on page 157, that the positive moments from a concentrated load are considerably larger than the negative moments, and by giving the arch another shape it is possible so to decrease the positive moments and increase the negative moments that they become equal, or so to form the arch that the moments from uniform loads and concentrated loads are reduced to a mean value of the separate moments; but only experts in the design of arches can do this and it takes a great amount of time, and the cost of the design may be considerably higher than the saving in material.

The bridges as given in the tables are considerably lighter than ordinarily built in the United States. There exist in Europe bridges which are considerably lighter still, which were tested by competent government officials for concentrated and uniform loads, and which gave no trouble in twenty years.

The quantities of the concrete and reinforcement per lineal foot width of the arch is simply found by the following formula: Let d=thickness of arch at the crown in inches and 1½ d the thickness of the arch at the abutment, and $L = l\left(1+\frac{8}{3}\frac{f^2}{l^2}\right)$ approximately the length of the arch in feet, then the number of cubic feet per lineal foot width of the arch ring$=\frac{d\,L}{9.6}$ and the weight of the reinforcement (a total of 1% of the crown section as longitudinal reinforcement and a total of ¼% of the crown section as transverse reinforcement)$=0.60\ d\ L$ pounds, including overlaps.

The spandril walls may be built of reinforced concrete according to Type A, given on page 116, or may be built of brick, rubble stone, cut stone, etc.

The fill in cubic feet$=\frac{f\,l}{3}$ per lineal foot width of bridge.

The cost of the abutments of an arch bridge is often as great as that of the arch proper and may be even considerable greater on bad ground, while on rock the cost is inconsiderable.

The design of the abutments is very simple. In our example the horizontal projection of the thrust was 65000. The direction of the thrust at the abutment is given by

$$\mathrm{tg}\ b = \frac{8f}{l}\,\frac{1+3\frac{a}{f}}{1+6\frac{a}{f}}$$

; a : f is about 0.2, or tg b=0.97, and the thrust =62000÷cos. b=90600 pounds. This thrust and the weight per lineal foot of the abutment of 32200 pounds (calculated from Figure 76) and the weight per lineal foot of the earth above it 64000 pounds, gives a resultant of 176,000 pounds, which strikes the base 3′ from the center. The component under right angles to the base=174,000, and the direct compressive stress per square foot=174,000÷20 (width of abutment)=8700 pounds per square foot while the stresses in the extreme points of the base are found by $174{,}000 \times 3 \div S\ \frac{1}{6}\ 20^2 = 7900$ pounds per square foot, resulting in a distribution of stresses as shown in diagram in Figure 76. A marked economy results from giving the base of the abutment an inclination of about 1 : 4, but in order to prevent sliding before the bridge is finished a small portion of the base should be made horizontal.

Piers carrying an arch on each side do not need, for the arch section alone, to be made wider than three, at the utmost four, times the thickness of the arch at the crown and the foundation of the piers need only to be figured for the vertical loads and a moment per lineal foot width=$100 \frac{l^2 h}{8f}$ when l the span in feet, f the rise of the arch and h the height of the pier in feet.

The design of a bridge with ribbed arches is illustrated in Figure 77, for a bridge of 250′ span and a rise of 40′.

The tables on pages 177-179 give the values of $d \times c/3$, the thrust from dead load and ½ live load for six feet width of the bridge (it being assumed that the ribs are 6′ c. c.) and the area for six feet width of bridge. The capital letters indicate the type of rib as given on page 168. The thrust is found by formula on page 153, and the bridges of 200′ span or more need only to be figured for a moving uniform load, as this is more unfavorable than a 24-ton wagon according to Cooper's specifications.

In our example T=630,000, and the bending moment 375,000 foot-pounds; the moment of resistance, according to page 168,=17700 inch^3; the area=1915 square inches; therefore the compressive stress from thrust=630,000÷1915=329 pounds per square inch, and the stresses in the extreme fibres from the bending moment$=375{,}000\times 12\div 17700=255$ pounds per square inch, or a total stress of 584 pounds per square inch. The bending moment from a change in temperature $=940\times\frac{I}{f}=940\times\frac{513000}{40\times 12}=1{,}000{,}000$ inch-pounds, or the stress in the extreme fibres$=1{,}000{,}000\div 17700=56$ pounds per square inch.

The value of $d+c/3$ was obtained in the following way:

The weight of the pavement was assumed=100 pounds per square foot, the weight of the reinforced concrete bridge floor including columns=100 pounds per square foot, half the live load=50 pounds, the weight of the ribs including slab for a width of six feet=1536 pounds; hence $d=1536+6\times 250=3036$ pounds. We assume that only the rib of a clear depth of 40″ increases to a clear depth of 60″ at the abutment, or the increases of weight at the abutment$=20\times 24=480$ pounds (one square inch of concrete one foot long weighs one pound), or $c/3=160$ pounds or $d+c/3=3196$ pounds.

The thrust at the abutment=730,000 pounds and the resultant of the thrust and the weight of the earth on the base of the abutment of a width of 6′ (=1, 400,000 pounds)= 1, 900,000 pounds and intersects the base (length of base =40′) 2′—3″ from its center. The component under right angle to the base=1, 880,000 pounds and the moment about the center of base$=1{,}880{,}000\times 2.25$ and this must be equal to $S\times 6\times\frac{40^2}{6}$ or the stress in the extreme edges of the base S=2670 pounds per square foot. The direct compressive stress=1, 880,000 : $6\times 40=7850$ pounds per square foot or the combined stresses as given in diagram in Figure 77 are 10520 and 5180 pounds per square foot, respectively. If the earth pressure is considered, the moment about the center of the base can be found graphically=700,000 foot-pounds per lineal foot, or the stresses in the extreme edges of the base 2650 pounds per square foot. These stresses are in opposite direction of those of thrust, therefore we would have in this case a compression of 7970 pounds per square foot on the water side and a compression of 7830 at the land side of the abutment.

The abutment does not need to be solid, and may be designed similar to retaining walls with counterforts, a counterfort at each rib. We have yet to figure the floor construction, which carries the pavement. The columns supporting this floor are spaced 6" c. c. in transverse direction and 10′ to 15′ in longitudinal direction. For a 12-ton wagon and a longitudinal spacing of columns of 10′, we need No. 12 girders in both directions and a 5″ slab, reinforced by 0.2 square inches per lineal foot in transverse direction and by 0.15 square inches in longitudinal direction, requiring an average of 0.566 cubic feet of concrete and 3.39 pounds of steel per square foot. For a spacing of columns of 15′, we need No. 16 girders in longitudinal direction, while slab and cross girders may be the same as above, requiring 0.548 cubic feet of concrete and 4.15 pounds of steel per square foot.

For a 24-ton wagon and a spacing of 10′ of the columns we need No. 15 and No. 12 girders in longitudinal and transverse direction, respectively, and a 5″ slab, reinforced by 0.36 and 0.15 square inches of steel per lineal foot, requiring 0.566 cubic feet of concrete and 4.87 pounds of steel per square foot. For a spacing of 15′ feet we can use the same slab and cross girders, but the longitudinal girders must be No. 25, requiring 0.566 cubic feet of concrete, 5.25 pounds of steel per square foot.

The supporting columns have to carry an area of only 60 to 90 square feet and a load not exceeding 25000 to 30000 pounds, hence 10″, or at the utmost 12″, square columns, reinforced by four ¾″ round bars, are amply strong.

The table on page 169 gives the property of arch ribs without connecting slab, and the use of such ribs is of importance where the traffic of the bridge is light, as for example in public parks, and where the foundations for the abutments would prove too expensive for a heavy bridge.

For an example we shall assume that we have to design a bridge of 150′ span, 40′ foot rise and a width of 12′. After a short trial we find that two arched ribs 14″x14″, 6′ c. c., reinforced by 1% of steel both at the top and the bottom, will answer. The bridge will have a granitoid finish, and the weight of floor construction will not exceed 80 pounds per square foot. In this case $d+c/3=6\times80+6\times50+14\times14+44=1080$ pounds per lineal foot of rib and the thrust $=1080\times\frac{150^2}{8\text{x}40}=76000$ pounds and the greatest moment from a uniform load $=600\times\frac{150^2}{100}=135{,}000$ foot-pounds. The area of the rib $=255$ square inches and the moment of resistance $=711$ inch^3, and the direct compressive stress $76000\div255=297$ pounds and the stress in the extreme fibres from the bending moment $=135{,}000\times12\div711=228$ pounds per square inch, giving a maximum stress of 525 pounds. The ribs should be connected by three stiffeners about 14"x14" at each ¼ point of the arch. In the entire bridge, inclusive of floor construction, are 2000 cubic feet of concrete and 12500 pounds of steel.

Form Lumber: The amount of lumber required for the form work of a solid arch or a ribbed arch with slab (exclusive of the lumber for the reinforced concrete floor carrying the pavement) varies from 4 to 6 feet B. M. per square foot of bridge for the intrados, and from 0.1 to 0.25 feet B. M. times the area in square feet between the intrados and the bed of the stream or ravine, for the supports and bracing, for each lineal foot of the width of the bridge.

PROPERTIES OF ARCH RINGS, 12″ WIDE AND REINFORCED BY ¼% AND ½% AT TOP AND BOTTOM.

Thickness of Slab Inches	Area of Plain Slab 12″ Wide in Square Inches	Reinforcement ¼% at Top and ¼% at Bottom			Reinforcemnnt ½% at Top and ½% at Bottom		
		Moment of Inertia Inch^4	Moment of Resistance Inch^3	Area Sq. In.	Moment of Inertia Inch^4	Moment of Resistance Inch^3	Area Sq. In.
4	48	66	33.	51.6	68	34.0	55
5	60	132	53.	64.5	139	55.8	69
6	72	232	77.	74.4	249	83.0	83
7	84	375	107.	90.3	407	116.0	96
8	96	566	141.	103.2	621	156.	110
9	108	814	181.	116.1	900	200.	124
10	120	1126	225.	129.0	1252	250.	138
11	132	1509	274.	141.9	1688	306.	152
12	144	1972	329.	154.8	2216	370.	165
13	156	2519	387.	157.7	2841	453.	180
14	168	3161	452.	180.6	3577	511.	193
15	180	3906	521.	193.5	4437	591.	207
16	192	4750	594.	206.4	5404	676.	221
17	204	5713	672.	219.3	6513	770.	234
18	216	6807	760.	232.2	7782	865.	248
19	228	8019	845.	245.1	9179	966.	262
20	240	9380	938.	258.	10750	1075.	276
21	252	10881	1040.	270.9	12501	1190.	290
22	264	12530	1140.	283.8	14410	1310.	304
23	276	14337	1240.	295.7	16597	1440.	317
24	288	16204	1350.	309.6	18584	1540.	331
25	300	18350	1470.	322.5	21075	1690.	345
26	312	20676	1590.	335.4	23776	1820.	358
27	324	23183	1720.	348.3	26683	1980.	372
30	360	31920	2120.	387.0	36840	2450.	414

Up to 23″ it was assumed that the center of reinforcement is 1¼″ from each face and 1½″ for the thicker slabs.

The area, moment of inertia and moment of resistance of the composite section was found by assuming E steel : E concrete=15.

PROPERTIES OF ARCH RIBS WITH CONNECTING SLAB, FOR A WIDTH OF SIX FEET.

Type	Detonations in Figures below			Area of Reinforcement at Bottom = Area at Top Sq. In.	Area of Plain Concrete Section	Area of Section including 15 times Area of Reinforcement	Moment at Inertia in 1000 Inch^4	Moment at Resistance in 1000 Inch^3
	d"	w"	t"					
A	36	20	5	7.5	980	1205	178	8.5
B	42	24	6	7.5	1296	1521	298	12.0
C	42	24	6	10	1296	1596	324	13.0
D	48	24	8	8.75	1536	1798	461	15.9
E	48	24	8	10	1536	1836	478	16.5
F	48	24	8	12.5	1956	1915	513	17.7
G	48	24	8	15	1536	1986	548	18.9
H	54	30	8	12.5	1956	2331	765	24.7
I	54	30	8	15	1956	2406	809	26.1
J	60	30	8	15	2136	2586	1069	31.2
K	60	30	8	17.5	2136	2661	1123	33.0
L	70	30	8	20	2436	3036	1761	45.2
M*	70	30	10	15	2940	3390	2075	59.0
N*	80	30	10	15	3240	3690	2901	72.9
O*	80	30	10	20	3240	3840	3096	77.2
P*	80	30	10	25	3240	3990	3286	82.1
Q*	80	30	15	30	3660	4560	3804	95.0
R*	90	30	15	25	3960	4710	4871	108.0
S*	100	30	15	50	4660	6160	7970	159.4
T*	120	40	15	50	5760	7260	13160	219.0
U*	140	40	15	50	6560	8060	19500	279.0

* Denotes that the section is as Figure No. 79.

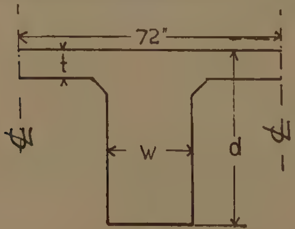


Fig. 78.

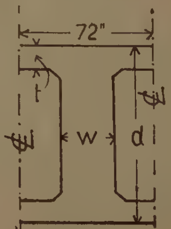


Fig. 79.

PROPERTIES OF ARCH RIBS WITHOUT CONNECTING SLABS.

Width and Depth of Rib Inches	Area of Plain Section Square Inches	Reinforcement 1% Top and 1% Bottom			Reinforcement 1½% Top and 1½% Bottom		
		Moment of Inertia Inch^4	Moment of Resistance Inch^3	Area Sq. Inches	Moment of Inertia Inch^4	Moment of Resistance Inch^3	Area Sq. Inches
8x 8	64	461	115	83	521	130	93
8x12	96	1732	289	125	2022	337	139
10x10	100	1100	220	130	1233	246	145
10x12	120	2169	361	156	2530	422	174
10x16	160	5430	680	208	6450	807	232
12x12	144	2603	435	187	3038	506	209
12x16	192	6516	815	250	7736	972	278
12x20	240	13200	1320	312	15800	1580	348
12x24	288	23374	1940	375	28124	2348	418
14x14	196	4980	711	255	5870	839	283
14x20	280	15410	1541	364	18450	1845	406
14x24	336	27300	2280	436	32900	2750	487
16x16	256	8690	1085	333	10300	1290	372
16x24	384	31100	2590	500	36400	3030	557
16x30	480	62200	4150	624	75300	5020	696
16x36	576	109300	6070	749	132800	7390	830
20x20	400	21970	2197	520	26300	2630	580
20x24	480	39000	3250	624	47000	3920	696
20x30	600	77800	5200	780	94100	6290	870
20x36	720	133000	7400	936	160700	8900	1044
20x42	840	214400	10200	1092	259900	12400	1218
24x24	576	44820	3730	749	53410	4350	830
24x30	720	90500	6030	936	108750	7210	1044
24x36	864	159800	8840	1120	193000	10750	1250
24x42	1008	256100	12200	1311	310100	14800	1461
30x36	1080	199000	11050	1400	240500	13400	1556
30x48	1440	485000	20200	1870	590000	24600	2088
30x60	1800	963000	32100	2340	1174500	39100	2610

In the ribs up to 20"x30" the distance of the center of reinforcement to each face was assumed=½"; in the ribs below that, 2".

DESIGN OF SOLID ARCH BRIDGES.

SPAN	RISE IN FEET								
30'	**3'**	**4'**	**5'**	**6'**	**7'**	**8'**	**10'**	**12'**	**15'**
a ÷ f	.75	.56	.45	.375	.32	.281	.225	.188	.15
T per Lin. Ft. Width	10300	7550	6950	6100	5450	5050	4550	4000	3560
$\frac{pl^2}{100}$ per Lin. Ft. Width	900	900	900	900	900	900	900	900	900
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	3750	2800	2250	1880	1610	1400	1120	930	750
M from 12 Ton Wagon	2140	2140	2140	2140	2140	2140	2140	2140	2140
T from 12 Ton Wagan	2000	1500	1200	1000	857	750	600	500	400
M from 24 Ton Wagon	4280	4280	4280	4280	4280	4280	4280	4280	4280
T from 24 Ton Wagon	4000	3000	2400	2000	1714	1500	1200	1000	800
Thickness at Crown For Unif. Load	4"				4"				4"
For 12 ton Wagon	6"				5"				5"
For 24 ton Wagon	7"				7"				7"
40'	**4'**	**5'**	**6'**	**7'**	**8'**	**9'**	**10'**	**12'**	**15'**
a ÷ f	.575	.46	.383	.330	.288	.256	.230	.192	.154
T per Lin. Ft. Width	15000	12500	11000	9950	9100	8450	7920	7190	6400
$\frac{pl^2}{100}$ per Lin. Ft. Width	1600	1600	1600	1600	1600	1600	1600	1600	1600
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	5000	4000	3330	2850	2500	2220	2000	1680	1330
M from 12 Ton Wagon	2850	2850	2850	2850	2850	2850	2850	2850	2850
T from 12 Ton Wagon	2000	1600	1330	1140	1000	890	800	667	533
M from 24 Ton Wagon	5700	5700	5700	5700	5700	5700	5700	5700	5700
T from 24 Ton Wogon	4000	3200	2660	2280	2000	1780	1600	1334	1066
Thickness at Crown For Unif. Load	5"				5"				5"
For 12 ton Wagon	7"				6"				6"
For 24 ton Wagon	9"				8"				8"
50'	**5'**	**6'**	**7'**	**8'**	**9'**	**10'**	**12'**	**15'**	**20'**
a ÷ f	.5	.416	.357	.3125	.279	.25	.208	.167	.125
T per Lin. Ft. Width	20900	18200	16400	15000	13800	13000	11700	10400	9150
$\frac{pl^2}{100}$ per Lin. Ft. Width	2500	2500	2500	2500	2500	2500	2500	2500	2500
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	6250	5200	4450	3900	3460	3120	2600	2080	1560
M from 12 Ton Wagon	3760	3760	3760	3760	3760	3760	3760	3760	3760
T from 12 Ton Wagon	2000	1670	1430	1250	1110	1000	836	666	500
M from 24 Ton Wagon	7520	7520	7520	7520	7520	7520	7520	7520	7520
T from 24 Ton Wagon	4000	3340	2860	2500	2220	2000	1672	1332	1000
Thickness at Crown For Unif. Load	7"				7"				6"
For 12 ton Wagon	8"				7"				7"
For 24 ton Wagon	10"				9"				9"

DESIGN OF SOLID ARCH BRIDGES.

SPAN	RISE IN FEET								
60'	**6'**	**7'**	**8'**	**9'**	**10'**	**12'**	**15'**	**20'**	**25'**
a ÷ f	.46	.392	.345	.305	.275	.230	.183	.1375	.110
T per Lin. Ft. Width	28200	25100	23700	21300	20000	17900	15800	13700	12500
$\frac{pl^2}{100}$ per Lin. Ft. Width	3600	3600	3600	3600	3600	3600	3600	3600	3600
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	9000	6400	5600	5000	4500	3750	3000	2250	1800
M from 12 Ton Wagon	5020	5020	5020	5020	5020	5020	5020	5020	5020
T from 12 Ton Wagon	2240	1920	1680	1500	1350	1120	895	672	538
M from 24 Ton Wagon	10040	10040	10040	10040	10040	10040	10040	10040	10040
T from 24 Ton Wagon	4480	3840	3360	3000	2700	2240	1790	1354	1076
Thickness at Crown: For Unif. Load	8"					8"			7"
For 12 ton Wagon	9"					9"			8"
For 24 ton Wagon	12"					11"			11"
70'	**7'**	**8'**	**9'**	**10'**	**11'**	**12'**	**15'**	**20'**	**30'**
a ÷ f	.40	.35	.311	.28	.254	.233	.187	.140	.093
T per Lin. Ft. Width	34600	31600	29200	27400	25700	24500	21700	18800	16800
$\frac{pl^2}{100}$ per Lin. Ft. Width	4900	4900	4900	4900	4900	4900	4900	4900	4900
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	8800	7700	6800	6150	5560	5100	4100	3070	2050
M from 12 Ton Wagon	6300	6300	6300	6300	6300	6300	6300	6300	6300
T from 12 Ton Wagon	2400	2100	1870	1680	1530	1400	1120	840	560
M from 24 Ton Wagon	12600	12600	12600	12600	12600	12600	12600	12600	12600
T from 24 Ton Wagon	4800	4200	3740	3360	3060	2800	2240	1680	1120
Thickness at Crown: For Unif. Load	10"					9"			8"
For 12 ton Wagon	11"					10"			9"
For 24 ton Wagon	13"					12"			12"
80'	**8'**	**9'**	**10'**	**11'**	**12'**	**13'**	**15'**	**20'**	**30'**
a ÷ f	.388	.344	.310	·281	.258	.239	.207	.156	.104
T per Lin. Ft. Width	44500	40900	38100	35800	34000	32400	29800	25800	21600
$\frac{pl^2}{100}$ per Lin. Ft. Width	6400	6400	6400	6400	6400	6400	6400	6400	6400
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	10000	8900	8000	7250	6670	6150	5330	4000	2670
M from 12 Ton Wagon	7700	7700	7700	7700	7700	7700	7700	7700	7700
T from 12 Ton Wagon	2560	2280	2050	1860	1710	1580	1370	1030	685
M from 24 Ton Wagon	15400	15400	15400	15400	15400	15400	15400	15400	15400
T from 24 Ton Wagon	5120	4560	4100	3720	3420	3160	2740	2060	1370
Thickness at Crown: For Unif. Load	11"						10"		10"
For 12 ton Wagon	12"						11"		11"
For 24 ton Wagon	15"						14"		14"

DESIGN OF SOLID ARCH BRIDGES.

Span	RISE IN FEET								
90'	9'	10'	11'	12'	13'	15'	20'	25'	30'
a ÷ f	.361	.325	.295	.271	.250	.216	.163	.130	.108
T per Lin. Ft. Width	53500	49700	46500	43400	42200	38700	33400	30000	27800
$\frac{pl^2}{100}$ per Lin. Ft. Width	8100	8100	8100	8100	8100	8100	8100	8100	8100
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	11400	10200	9300	8500	7850	6800	5100	4100	3400
M from 24 Ton Wagon	17800	17800	17800	17800	17800	17800	17800	17800	17800
T from 24 Ton Wagon	5280	4740	4280	3940	3640	3160	2380	1900	1580
Thickness at Crown: For Unif. Load	13"					12"			11"
Thickness at Crown: For 24 ton Wagon	16"					15"			14"

Span	RISE IN FEET								
100'	10'	11'	12'	13'	15'	20'	25'	30'	35'
a ÷ f	.340	.31	.283	.262	.226	.170	.136	.114	.0975
T per Lin. Ft. Width	63400	59600	56300	53600	49100	42100	37900	35100	33000
$\frac{pl^2}{100}$ per Lin. Ft. Width	10000	10000	10000	10000	10000	10000	10000	10000	10000
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	12500	11400	10400	9600	8300	6250	5000	4150	3550
M from 24 Ton Wagon	20400	20400	20400	20400	20400	20400	20400	20400	20400
T from 24 Ton Wagon	5440	4540	4320	4180	3640	2720	2180	1800	1540
Thickness at Crown: For Unif. Load	15"					13"			12"
Thickness at Crown: For 24 ton Wagon	17"					16"			15"

DESIGN OF SOLID ARCH BRIDGES.

Span	RISE IN FEET								
110'	10'	11'	12'	13'	15'	20'	25'	30'	35'
a ÷ f	.35	.318	.291	.270	.233	.175	.140	.117	.10
T per Lin. Ft. Width	78000	73200	69300	66100	60500	51900	46500	43000	40300
$\frac{pl^2}{100}$ per Lin. Ft. Width	12100	12100	12100	12100	12100	12100	12100	12100	12100
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	15100	13700	12600	11600	10000	7550	6000	5000	4300
M from 24 Ton Wagon	23000	23000	23000	23000	23000	23000	23000	23000	23000
T from 24 Ton Wagon	6140	5600	5120	4760	4100	3080	2460	2080	1760
Thickness at Crown: For Unif. Load	17"					15"			13"
Thickness at Crown: For 24 ton Wagon	19"					17"			16"

Span	RISE IN FEET								
120'	10'	11'	12'	13'	15'	20'	25'	30'	35'
a ÷ f	.39	.355	.325	.300	.260	.195	.156	.130	.110
T per Lin. Ft. Width	100000	94000	88500	84000	76900	65000	58000	53200	50000
$\frac{pl^2}{100}$ per Lin. Ft. Width	14400	14400	14400	14400	14400	14400	14400	14400	14400
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	18000	16400	15000	13800	12000	9000	7200	6000	5120
M from 24 Ton Wagon	26000	26000	26000	26000	26000	26000	26000	26000	26000
T from 24 Ton Wagon	6900	6350	5750	5320	4610	3450	2770	2300	1970
Thickness at Crown: For Unif. Load	19"					18"			17"
Thickness at Crown: For 24 ton Wagon	21"					20"			18"

DESIGN OF SOLID ARCH BRIDGES.

Span	RISE IN FEET								
130'	13'	14'	15'	16'	20'	25'	30'	35'	40'
$a \div f$	.317	.293	.273	.256	.205	.164	.137	.117	.103
T per Lin. Ft. Width	103000	97200	93000	89300	78500	70000	64200	60000	57100
$\frac{pl^2}{100}$ per Lin. Ft. Width	16900	16900	16900	16900	16900	16900	16900	16900	16900
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	16400	15200	14150	13250	10600	8500	7100	6050	5300
M from 24 Ton Wagon	28200	28200	28200	28200	28200	28200	28200	28200	28200
T from 24 Ton Wagon	6000	5500	5120	4800	3850	3060	2560	2200	1930
Thickness at Crown: For Unif. Load	20"					19"			18"
Thickness at Crown: For 24 ton Wagon	22"					21"			19"

Span	RISE IN FEET								
140'	14'	15'	16'	17'	20'	25'	30'	35'	40'
$a \div f$	.306	.286	.268	.252	.215	.172	.143	.123	.107
T per Lin. Ft. Width	115000	111000	106000	103000	93500	85700	76000	71000	67000
$\frac{pl^2}{100}$ per Lin. Ft. Width	19600	19600	19600	19600	19600	19600	19600	19600	19600
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	17500	16300	15350	14400	12250	9800	8150	7000	6150
M from 24 Ton Wagon	31100	31100	31100	31100	31100	31100	31100	31100	31100
T from 24 Ton Wagon	6000	5500	5200	4900	4150	3350	2750	2370	2080
Thickness at Crown: For Unif. Load	22"					20"			20"
Thickness at Crown: For 24 ton Wagon	24"					23"			21"

DESIGN OF SOLID ARCH BRIDGES.

Span	RISE IN FEET								
150'	15'	16'	17'	18'	20'	25'	30'	35'	40'
$\frac{pl^2}{100}$ per Lin. Ft. Width	22500	22500	22500	22500	22500	22500	22500	22500	22500
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	18800	17600	16600	15700	14100	11300	9400	8100	7060
T per Lin. Ft. Width...	102000	95500	90000	85000	76500	61700	51000	44000	38350
M from 24 Ton Wagon..	33500	33500	33500	33500	33500	33500	33500	33500	33500
T from 24 Ton Wagon..	6000	5600	5200	5000	4500	3600	3000	2560	2230
Thickness at Crown For Unif. Load..	22"					19"			18"
Thickness at Crown For 24 ton Wagon	24"					20"			19"

Span	RISE IN FEET								
160'	16'	17'	18'	19'	20'	25'	30'	35'	40'
$\frac{pl^2}{100}$ per Lin. Ft. Width	25600	25600	25600	25600	25600	25600	25600	25600	25600
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	20000	18800	17700	16800	16000	12700	10600	9100	8000
T per Lin. Ft. Width...	116000	111000	102000	97000	92500	74000	61300	52500	46700
M from 24 Ton Wagon..	36500	36500	36500	36500	36500	36500	36500	36500	36500
T from 24 Ton Wagon..	6100	5700	5400	5120	4870	3880	3250	2780	2440
Thickness at Crown For Unif. Load..	23"					20"			19"
Thickness at Crown For 24 ton Wagon	25"					22"			20"

DESIGN OF SOLID ARCH BRIDGES.

Span	RISE IN FEET								
170'	17'	18'	19'	20'	25'	30'	35'	40'	45'
$\frac{pl^2}{100}$ per Lin. Ft. Width	28900	28900	28900	28900	28900	28900	28900	28900	28900
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	21300	20100	19000	18100	14400	12100	10300	9000	8000
T per Lin. Ft. Width...	130000	122000	116000	110000	87500	73500	63000	54700	49000
M from 24 Ton Wagon..	39000	39000	39000	39000	39000	39000	39000	39000	39000
T from 24 Ton Wagon..	6200	5820	5500	5220	4970	3960	3300	2850	2490
Thickness at Crown: For Unif. Load..	24"					21"			20"
Thickness at Crown: For 24 ton Wagon	26"					22"			21"

Span	RISE IN FEET								
180'	18'	19'	20'	22'	25'	30'	35'	40'	45'
$\frac{pl^2}{100}$ per Lin. Ft. Width	32400	32400	32400	32400	32400	32400	32400	32400	32400
$100\frac{l^2}{8f}$ per Lin. Ft. Wd.	22500	21300	20300	18500	16200	13500	11600	10100	9000
T per Lin. Ft. Width...	143000	135000	129000	117000	103000	86000	73000	64000	57600
M from 24 Ton Wagon..	42000	42000	42000	42000	42000	42000	42000	42000	42000
T from 24 Ton Wagon..	6200	5820	5500	5220	4970	3960	3300	2850	2490
Thickness at Crown: For Unif. Load..	27"					22"			21"
Thickness at Crown: For 24 ton Wagon	30"					24"			22"

DESIGN OF AND STRESSES IN RIBBED ARCHES.

Span	200'				225'			
	Bending Moment Foot Pounds	Rise in Feet			Bending Moment Foot Pounds	Rise in Feet		
		20′	35′	60′		25′	40′	70′
$d + \frac{c}{3}$		3196	2964	2583		3726	3233	2965
Thrust for Six Foot Width	240000	800000	423000	216000	303600	940000	512000	268000
Area for Six Foot Width		1986	1522	1205		2406	1840	1521
Compressive Stress from Thrust		407	278	180		392	280	176
Stress from Bending Moment		153	240	341		140	220	303
Temperature Stress		117	40	32		126	58	29
Section at Crown Type		G	B	A		I	E	B

Span	250'				275'			
	Bending Moment Foot Pounds	Rise in Feet			Bending Moment Foot Pounds	Rise in Feet		
		25′	40′	70′		30′	60′	90′
$d + \frac{c}{3}$		3936	3196	2965		4256	3726	3233
Thrust for Six Foot Width	375000	1230000	630000	331000	455000	1340000	588000	340000
Area for Six Foot Width		2661	1915	1596		3036	2331	1802
Compressive Stress from Thrust		460	329	207		443	251	189
Stress from Bending Moment		137	255	346		121	221	343
Temperature Stress		110	56	30		102	36	26
Section at Crown Type		K	F	C		L	H	D

The type Letters refer to page 168.

DESIGN OF AND STRESSES IN RIBBED ARCH BRIDGES.

Span	300′				330′			
	Bending Moment Foot Pounds	Rise in Feet			Bending Moment Foot Pounds	Rise in Feet		
		30′	60′	90′		35′	70′	100′
$d + \frac{c}{3}$		4780	3726	3233		5040	3936	3727
Thrust for Six Foot Width	540000	1800000	700000	405000	650000	1950000	762000	508000
Area for Six Foot Width		3480	2406	1950		3840	2586	2406
Compressive Stress from Thrust		376	291	212		509	296	211
Stress from Bending Moment		110	248	343		101	250	300
Temperature Stress		92	42	25		90	39	26
Section at Crown Type		M	I	F		O	J	I

Span	360′				400′			
	Bending Moment Foot Pounds	Rise in Feet			Bending Moment Foot Pounds	Rise in Feet		
		40′	70′	100′		40′	70′	100′
$d + \frac{c}{3}$		5760	4780	3936		6585	5040	4780
Thrust for Six Foot Width	780000	2340000	1100000	640000	960000	3300000	1440000	956000
Area for Six Foot Width		4710	3480	2586		6160	3690	3480
Compressive Stress from Thrust		498	320	248		538	390	138
Stress from Bending Moment		87	159	300		73	158	195
Temperature Stress		87	40	28		99	43	28
Section at Crown Type		R	M	J		S	N	M

The type Letters refer to page 168.

DESIGN OF AND STRESSES IN RIBBED ARCH BRIDGES.

Span	430′				460′			
	Bending Moment Foot Pounds	Rise in Feet			Bending Moment Foot Pounds	Rise in Feet		
		50′	80′	110′		60′	80′	120′
$d + \frac{c}{3}$		7860	5040	4780		8810	5410	5410
Thrust for Six Foot Width	1160000	3620000	1460000	1010000	1270000	3870000	1790000	1200000
Area for Six Foot Width		7260	3840	3480		8060	4560	4260
Compressive Stress from Thrust		499	380	290		480	393	282
Stress from Bending Moment		64	180	236		55	160	197
Temperature Stress		95	42	26		91	45	29
Section at Crown Type		T	O	M		U	Q	O

Span	500′		
	Bending Moment Foot Pounds	Rise in Feet	
		70′	120′
$d + \frac{c}{3}$		8810	5040
Thrust for Six Foot Width	1500000	3940000	1320000
Area for Six Foot Width		8060	3990
Compressive Stress from Thrust		490	330
Stress from Bending Moment		65	220
Temperature Stress		79	27
Section at Crown Type		U	P

The type Letters refer to page 168.

STRESSES IN SOLID ARCH BRIDGES.

Span in Feet	Uniform Loading				12 Ton Wagon				24 Ton Wagon			
	Bending Moment Ft. Lbs.	Thrust for a Rise in Feet			Bending Moment Ft. Lbs.	Thrust for a Rise in Feet			Bending Moment Ft. Lbs.	Thrust for a Rise in Feet		
30'		3'	7'	15'		3'	7'	15'		3'	7'	15'
	900	10300	5450	3560	2140	11000	5500	3600	4280	12500	6360	3980
Thickness at Crown		4"	4"	4"		6"	5"	5"		7"	7"	7"
Compressive Stress from Thrust		188	99	65		132	80	52		130	66	41
Stress from Bending Moment		317	317	317		310	460	460		445	445	445
Temperature Stress		53	23	11		79	29	13		92	40	19
40'		4'	8'	15'		4'	8'	15'		4'	8'	15'
	1600	15000	9100	6400	2850	14500	9000	6300	5700	16500	9850	6800
Thickness at Crown		5"	5"	5"		7"	6"	6"		9"	8"	8"
Compressive Stress from Thrust		217	132	93		151	108	76		134	90	62
Stress from Bending Moment		345	345	345		295	412	412		342	439	439
Temperature Stress		49	25	13		69	30	16		89	40	22
50'		5'	9'	20'		5'	9'	20'		5'	9'	20'
	2500	20900	13800	9150	3760	19800	13200	8400	7520	21200	14300	9350
Thickness at Crown		7"	7"	6"		8"	7"	7"		10"	9"	9"
Compressive Stress from Thrust		217	143	110		180	136	87		154	116	76
Stress from Bending Moment		258	258	362		290	390	390		362	450	450
Temperature Stress		54	30	12		63	31	14		79	40	18

STRESSES IN SOLID ARCH BRIDGES.

Span in Feet	Uniform Loading				12 Ton Wagon				24 Ton Wagon			
60'	Bending Moment Foot Lbs.	Thrust for a Rise in Feet			Bending Moment Foot Lbs.	Thrust for a Rise in Feet			Bending Moment Foot Lbs.	Thrust for a Rise		
		6'	12'	25'		6'	12'	25'		6'	12'	25'
	3600	28200	17900	12500	5020	26000	17100	12100	10040	28200	18200	12700
Thickness at Crown...............		8"	8"	7"		9"	9"	8"		12"	11"	11"
Compressive Stress from Thrust..		250	163	130		210	138	110		172	120	84
Stress from Bending Moment.....		276	276	372		301	301	390		324	395	395
Temperature Stress..............		53	27	11		59	29	13		80	37	18
70'		7'	12'	30'		7'	12'	30'		7'	12'	30'
	4900	34600	24500	15800	6300	32600	23400	25400	12600	35000	24800	16000
Thickness at Crown...............		10"	9"	8"		11"	10"	9"		13"	12"	12"
Compressive Stress from Thrust..		252	198	144		215	170	124		195	150	97
Stress from Bending Moment......		235	295	377		247	302	378		333	408	408
Temperature Stress..............		57	30	11		63	33	12		70	39	16
80'		8'	15'	30'		8'	15'	30'		8'	15'	30'
	6400	44500	29800	21600	7700	42000	28500	21000	15400	44600	29900	21700
Thickness at Crown...............		11"	10"	10"		12"	11"	11"		15"	14"	14"
Compressive Stress from Thrust..		293	215	157		255	188	138		216	155	112
Stress from Bending Moment......		251	307	307		250	302	302		313	362	362
Temperature Stress..............		54	27	13		59	29	15		73	37	19

STRESSES IN SOLID ARCH BRIDGES.

Span in Feet	Uniform Load and 12 Ton Wagon				24 Ton Wagon			
90'	Bending Moment Foot Lbs.	Thrust for a Rise in Feet			Bending Moment Foot Lbs.	Thrust for a Rise in Feet		
		9'	15'	30'		9'	15'	30'
	8100	53500	38700	27800	17800	53000	38400	27700
Thickness at Crown...		13"	12	11		16	15	14
Compressive Stress...		297	235	184		240	186	144
Stress from Bend'g. M.		214	263	318		315	362	420
Temperature Stress...		53	32	15		70	40	19
100'		10'	20'	35'		10'	20'	35'
	10000	63400	42100	33000	20400	62600	41700	32800
Thickness at Crown...		15"	13	12		17	16	15
Compressive Stress...		307	234	200		267	189	159
Stress from Bend'g. M.		203	265	325		318	360	413
Temperature Stress...		59	25	14		67	32	17
110'		10'	20'	35'		10'	20'	35'
	12100	78000	51900	40300	23000	76500	51200	39400
Thickness at Crown...		17"	15	13		19	17	16
Compressive Stress...		333	250	225		293	220	178
Stress from Bend'g. M.		187	245	320		287	358	407
Temperature Stress...		67	30	14		76	34	18
120'		10'	20'	35'		10'	20'	35'
	14400	100000	65000	50000	26000	98000	64000	49000
Thickness at Crown...		19"	18	17		21	20	18
Compressive Stress...		383	262	214		338	233	198
Stress from Bend'g. M.		179	200	225		263	291	361
Temperature Stress...		75	36	20		83	40	21
130'		13'	25'	40'		13'	25'	40'
	16900	103000	70000	57100	28200	101000	69000	56500
Thickness at Crown...		20"	19	18		22	21	19
Compressive Stress...		375	268	230		333	238	216
Stress from Bend'g. M.		189	210	235		260	285	350
Temperature Stress...		57	31	18		67	33	19

The stresses are in pounds per square inch.

STRESSES IN SOLID ARCH BRIDGES.

Span in Feet	Uniform Load and 12 Ton Wagon				24 Ton Wagon			
	Bending Moment Foot Lbs.	Thrust for a Rise in Feet			Bending Moment Foot Lbs.	Thrust for a Rise in Feet		
140'		14'	25'	40'		14'	25'	40'
	19600	116000	85700	67000	31100	114000	84100	66000
Thickness at Crown...		22"	20	20		24	23	21
Compressive Stress...		382	311	243		345	266	227
Stress from Bend'g. M.		180	218	218		237	260	314
Temperature Stress...		63	32	20		68	37	21
150'		15'	25'	40'		15'	25'	40'
	22500	102000	61700	38350	33500	97000	58300	36200
Thickness at Crown...		22"	19	18		24	20	19
Compressive Stress...		335	236	156		292	191	138
Stress from Bend'g. M.		206	280	315		255	307	415
Temperature Stress...		58	30	18		63	26	19
160'		16'	25'	40'		16'	25'	40'
	25600	116000	74000	46700	36500	110000	70000	44100
Thickness at Crown...		23"	20	19		25	22	20
Compressive Stress...		365	268	179		320	231	160
Stress from Bend'g. M.		213	286	319		256	335	406
Temperature Stress...		57	32	19		59	35	20
170'		17'	30'	45'		17'	30'	45'
	28900	130000	73500	49000	39000	123000	70000	46500
Thickness at Crown...		24"	21	20		26	22	21
Compressive Stress...		393	254	177		343	231	160
Stress from Bend'g. M.		198	291	323		253	357	393
Temperature Stress...		50	28	18		61	29	19
180'		18'	30'	45'		18'	30'	45'
	32400	143000	86000	57600	42000	135000	81500	54500
Thickness at Crown...		27"	22	21		30	24	22
Compressive Stress...		385	284	199		326	246	180
Stress from Bend'g. M.		194	296	326		203	324	385
Temperature Stress...		53	29	19		66	32	20

The stresses are in pounds per square inch.

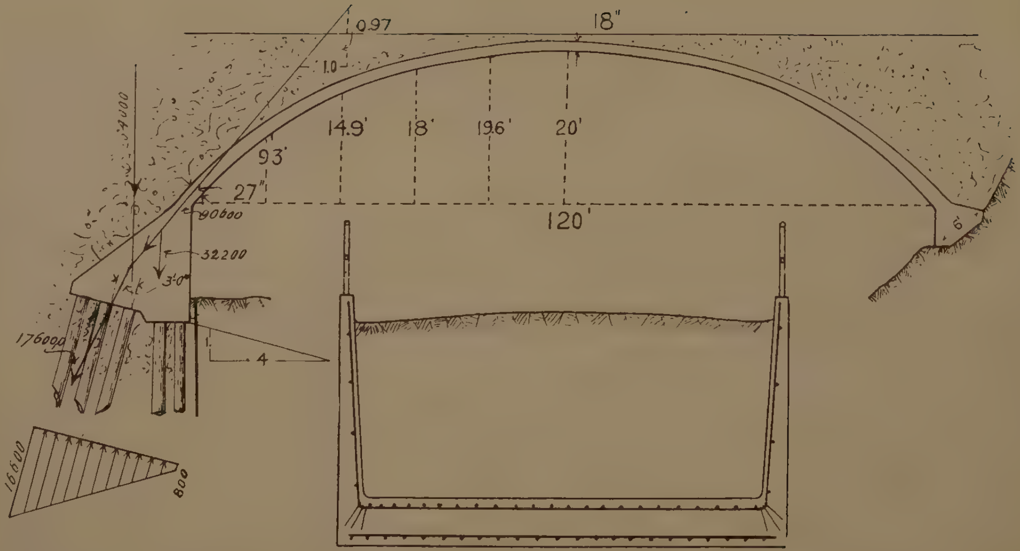


Fig. 76. Longitudinal and Cross Section of an Arched Bridge with fill and solid Spandrils. 120′ Span.

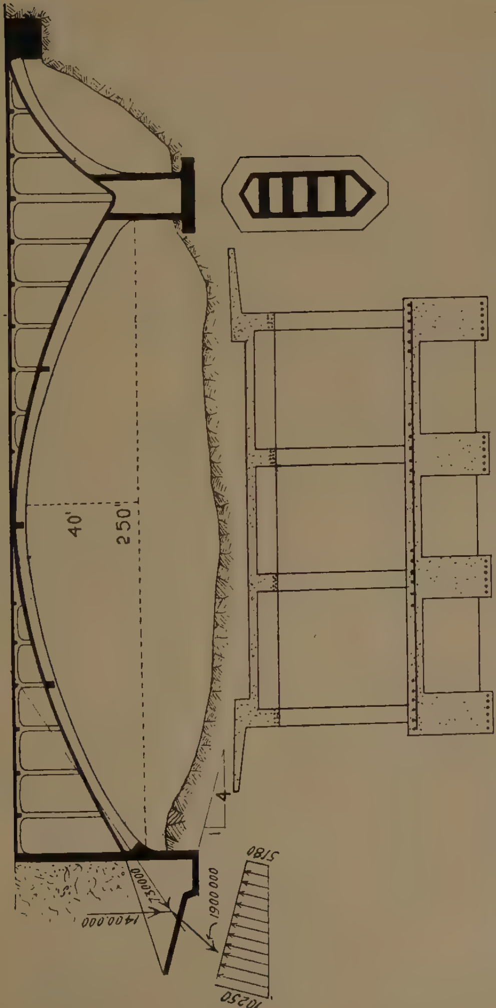


Fig. 77. Longitudinal and Cross Section of Ribbed Arch Bridge, open Spandril construction. 250′ Span.

GIRDER BRIDGES.

Reinforced concrete girder bridges are, as a rule, cheaper then arched bridges for the spans given in table on page 187.

We assume that the girders are six feet center to center. The thickness of the floor slab is governed by the requirement of the necessary flange area for the girders.

We gave in the first three lines of the table the equivalent uniform load which produces the same moment as a 20-ton steam roller, a 24-ton and 12-ton wagon, and as can be seen a 20-ton roller is more unfavorable than a 24-ton wagon. For the larger spans this equivalent load is not much greater than 600 pounds, which is the load from a crowd of people per lineal foot of 6′ width.

All the dimensions are clearly given in the table. The longitudinal reinforcement of the slab was assumed ¼%, in order to prevent cracks.

By connecting the main girders by one or two stiffening girders, the load is distributed over practically all girders in the bridge, and we can safely figure a capacity of the bridge from 25 to 40% greater than given in the table. These stiffening girders should be only a few inches less in depth than the main girders, 8″ to 12″ wide, and reinforced by ¼% at top and at the bottom.

The abutments can be made similar to the retaining walls given in pages 112-117, with the difference that the heel does not need to be as wide, on account of the additional weight of the bridge on the base, and that pilasters must be provided to offer a bearing to the bridge girders. This bearing should not be less than 12″ deep, and be better 18 to 24″ deep in the larger spans.

The sidewalks are generally in cantilever of a length of 2′-6″ to 3′-0″.

If the bridge girders are continuous, a reduction of about 20 to 25% can be made for the moment in the center, but the top of the beams must be heavily reinforced over the supports for the negative moments.

GIRDER BRIDGES.

Span in Feet	15′	20′	25′	30′	35′	40′	45′	50′	55′	60′	
LIVE LOAD PER LIN. FT. OF GIRDER											
Equivalent for a 20 Ton Steam Roller	1430	1290	1190	1100	1010	960	900	860	830	800	Lbs.
Equivalent for a 24 Ton Wagon	1560	1320	1160	1050	970	900	870	840	800	760	Lbs.
Equivalent for a 12 Ton Wagon	1000	820	800	720	680	660	625	600	600	600	Lbs.
Dead Load per Lineal Foot	1100	1200	1230	1300	1450	1550	1700	1800	1900	1980	Lbs.
Total Equivalent load for a 20 Ton Roller	2660	2520	2390	2350	2420	2450	2570	2640	2700	2740	Lbs.
Total Equivalent load for a 12 Ton Roller	2100	2020	2030	2020	2130	2310	2325	2400	2500	2580	Lbs.
Maximum Moment for a 20 Ton Roller	75	126	187	255	370	490	650	825	1020	1230	In 1000 Ft. Lbs.
Maximum Moment for a 12 Ton Roller	59	101	159	227	326	462	590	750	945	1160	In 1000 Ft. Lbs.
Beam Number for a 20 Ton Roller	27	55	62	68	77	84	88	91	96	97	
Beam Number for a 12 Ton Wagon	26	48	58	67	76	84	87	88	92	96	
Thickness of Slab	5″	5″	5″	5″	$5\frac{1}{2}$″	6″	7″	7″	7″	8″	
Sec. Area of Slab Reinforc't per lin. ft. width, Crosswise	.48	.48	.48	.48	.40	.32	.21	.21	.21	.24	
Sec. Area of Slab Reinforc't per lin. ft. width, Lengthwise	.15	.15	.15	.15	.165	.18	.21	.21	.21	.24	
Average Concrete per Sq. Ft., Cu. Ft.	.56	.64	.72	.77	.90	1.07	1.25	1.43	1.50	1.69	20 Ton Roller
Average Steel per Sq. Ft., Lbs.	6.2	6.6	7.7	8.2	9.6	9.8	11.0	11.0	12.6	15.6	
Average Form Lumber, Except Supports	2.8	3.4	3.45	3.10	3.20	3.6	3.85	4.10	4.20	4.40	
Average Concrete per Sq. Ft., Cu. Ft.	.56	.62	.69	.77	.90	1.07	1.25	1.25	1.43	1.58	12 Ton Wagon
Average Steel per Sq. Ft., Lbs.	5.6	6.2	7.2	7.7	8.9	9.8	9.40	11.0	12.60	12.90	
Average Form Lumber per Sq. Ft.	2.8	3.4	3.45	3.10	3.20	3.6	3.85	3.85	4.10	4.20	

Girders are 6′ c. c. Beam numbers refer to pages 6 to 10.

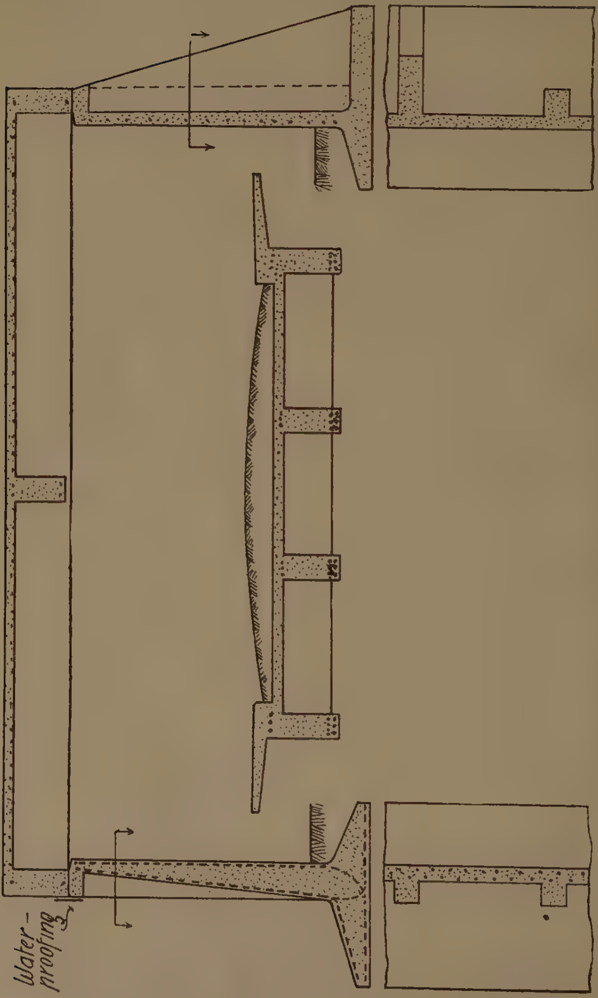


Fig. 80. Longitudinal and Cross Section of Girder Bridge. 40' Span.

FRAMED STRUCTURES.

The monolithic character of reinforced concrete construction unites the various members of construction to such an extent that the deflection in one member affects the deflection in a more or less degree in all other members.

The stresses become indeterminate, and can only be exactly found in the simplest cases.

Figure 81 represents a girder of the span l and the moment of inertia I_1, connected to two uprights of the height h and the moment of inertia I, and loaded by a uniform load p per lineal foot. Figure 82 represents the same structure, with no load on the girder, but affected by a triangular load, such as earth pressure, at the supports. Figure 83 represents the same structure acted upon by wind pressure on one side only.

Let $n = \frac{h}{l} \frac{I_1}{I}$, then by the theory of least work, the statically indeterminate value of Ma and T for uniform load and a concentrated load P in the center are given for Figure 81 by the formulae:

$$\left.\begin{aligned} T &= \frac{pl^2}{8h}\frac{1}{1+\frac{1}{2}n} \\ Ma &= \frac{pl^2}{12}\frac{1}{1+\frac{1}{2}n} \end{aligned}\right\} \text{for uniform load} \qquad \left.\begin{aligned} T &= \tfrac{3}{16}\frac{Pl}{h}\frac{1}{1+\frac{1}{2}n} \\ Ma &= \tfrac{1}{16}\frac{Pl}{h}\frac{1}{1+\frac{1}{2}n} \end{aligned}\right\} \text{for a concentrated load}$$

for Figure 82, when $E = \frac{p}{2}h^2$
$$\begin{cases} T = E x \tfrac{4}{5} x \dfrac{1+\frac{7}{16}n}{1+\frac{1}{2}n}, \\ Ma = E x \tfrac{4}{15} h x \dfrac{1+\frac{15}{80}n}{1+\frac{1}{2}n}, \end{cases}$$

for Figure 83, when p the pressure per lineal foot of support, the indeterminate values can be found by the equations,

$$Th\,(1+\tfrac{2}{3}n) - \frac{Ma}{2}(1+n) + \frac{Mb}{2}(1+n) = \tfrac{3}{4}ph^2\,(1+\tfrac{11}{18}n)$$

$$-\frac{Th}{2}(1+n) + \frac{Ma}{3}(1+3n) - \frac{Mb}{6} = -\tfrac{1}{3}ph^2\,(1+\tfrac{1}{2}n)$$

$$\frac{Th}{2}(1+n) - \frac{Ma}{6} + \frac{Mb}{3}(1+3n) = \tfrac{5}{12}ph^2\,(1+\tfrac{6}{5}n)$$

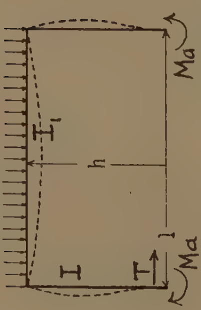


Fig. 81.

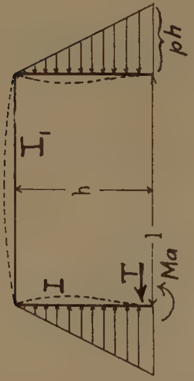


Fig. 82.

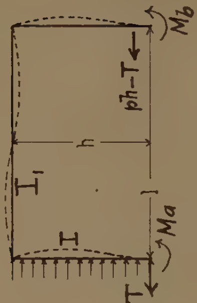


Fig. 83.

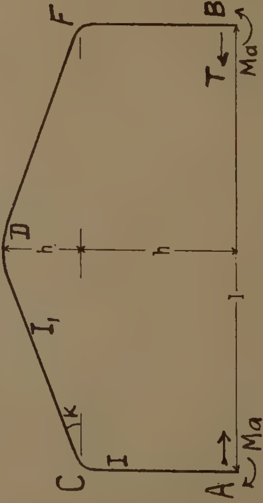


Fig. 84.

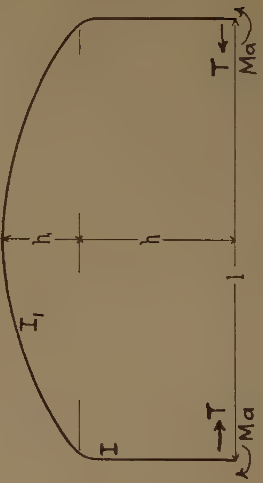


Fig. 85.

Figure 84 represents a roof girder of the moment of inertia of I_1, supported by two columns.

The indeterminate values of T and Ma can be found by solving the equations,

$$-Ma(1+\tfrac{1}{2}\frac{h_1}{h}+n)+Th(1+\frac{h_1}{h}+\tfrac{1}{3}\frac{{h_1}^2}{h^2}+\tfrac{2}{3}n)=\frac{pl^2}{12}(1+\tfrac{5}{8}\frac{h_1}{h})$$

$$+Ma\ (1+2n)-Th\ (1+\tfrac{1}{2}\frac{h_1}{h}+n)=-\frac{pl^2}{12}$$ when $n=\frac{h}{l}\ \frac{I_1}{I}$ $\times$ cos. k.

Figure 85 represents a parabolic roof girder on two supports. T and Ma may be found by solving,

$$Th(1+\tfrac{4}{3}\frac{h_1}{h}+\tfrac{8}{15}\frac{{h_1}^2}{h^2}+\tfrac{2}{3}n)-Ma\,(1+\tfrac{2}{3}\frac{h_1}{h}+n)=\frac{pl^2}{12}(1+\tfrac{4}{5}\frac{h_1}{h})$$

$$Ma\ (1+2n)-Th\ (1+\tfrac{2}{3}\frac{h_1}{h}+n)=-\frac{pl^2}{12}$$

The case represented by Figures 81 and 82 is encountered in all culverts and also in girder bridges, when the girders are monolithically connected with the abutments.

As an example we shall figure the shed construction as shown in Figure 86.

$h_1=14.75'$, $h=28.'$, $l=80'$, $I_1=18\,x\frac{45^3}{12}$, $I=18x\frac{65^3}{12}$

and $\frac{h_1}{h}=0.51$, $n=0.1095$, and the two equations are

$$-Ma(1+0.255+0.1095)+Th(1+0.255+0.0867+0.0735)=$$

$$\frac{pl^2}{12}(1+0.319)$$

$$Ma(1+0.2190)-Th(1+0.255+0.1095)=-\frac{pl^2}{12}$$

or $Ma=0.0175pl^2$ and $Th=0.0738pl^2$.

p, we assume=2000 pounds per lineal foot, and $pl^2=12{,}800{,}000$ and Ma=224,000 foot-pounds and Th=945,000 foot-pounds, or the moment at C=945,000—224,000=721,000 foot-pounds.

The moment at the center of the roof girder$=\frac{2000x80^2}{8}+$ $224000-\frac{945000}{h}(h+h_1)=396000$ foot-pounds.

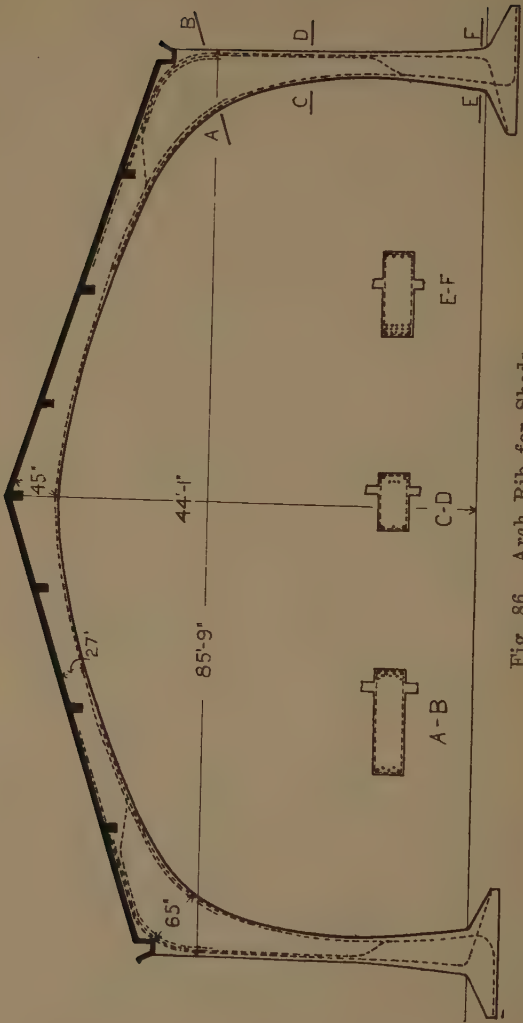


Fig. 86. Arch Rib for Sheds.

The wind stresses can be found, approximately, by using the equations given for Figure 83, which become in this case,

$$1.073\ Th - 0.55475\ Ma + 0.55475\ Mb = 0.800175\ ph^2$$
$$0.55475\ Th + 0.4428\ \ Ma - 0.16667\ Mb = -0.35158\ ph^2$$
$$0.55475\ Th - 0.16667\ Ma + 0.4428\ \ Mb = 0.471417\ ph^2$$

or $Th = 0.797\ ph^2$, $Ma = 0.269\ ph^2$, $Mb = 0.164\ ph^2$.

Assuming the ribs to be 16′ c.c, and the wind pressure=20 pounds per square foot, p=320 and $ph^2 = 253,000$.
or Th=202,000 foot-pounds, Ma=68000 foot-pounds, Mb= 41500 foot-pounds.

The thrust at the point B=ph—0.797 ph=0.203 ph=1800 pounds.

This moment Ma and thrust T have the tendency to diminish the stresses on the wind side, while the stresses at the lee side are increased.

The greatest moment at the base is at B=224,000+41500= 265,500 foot-pounds, and the greatest moment at C=721,000— 202,000+68000+320× $\frac{28^2}{2}$=712,000 foot-pounds, while at F the moment=721,000+1800×28—41500=730,000 foot-pounds.

The rib at F is 18″ wide and 65″ deep, or the coefficient= $\frac{730000}{1.5 \times 65^2}$=1.15, corresponding to a percentage of 1.06, or 12.5 square inches of steel. If the slab in the center of roof is omitted for the purpose of a lantern, we have to place in the center of the roof girder ten square inches of reinforcement, otherwise six square inches will suffice.

At the base the rib is 50″ wide, and we have to place 4.5 square inches of reinforcement at the inside face of the rib.

The footing of the ribs must be designed to take up the moment of 265,500 foot-pounds and the vertical load from the structure. Only compressive stresses can be produced on the ground and the vertical load must be large enough to prevent sliding of the base from the thrust action. The vertical load at the base = 2000 × 82.8 + weight of reinforced concrete wall (5″ thick) between the ribs and the weight of footing and dirt above it = 137,000 pounds.

Assuming the footing to be 6′ wide and 12′ long (in transverse direction of the shed), we have $265{,}500 = S6\text{x}\frac{12^2}{6}$ or the compressive stresses at the edges $S = 1840$ pounds per square foot and the direct compressive stresses $= \frac{137000}{6\text{x}12} = 1905$ pounds per square foot, or 3745 and 65 pounds are the compressive stresses at the edges of the base.

For another example we will assume a culvert of 8′ span and a height of 8′; floor and wall we assume of the same thickness, and as span and height are the same $n = 1$. For a uniform load of 1000 pounds per square foot $T = 1000 \times \frac{8^2}{8\text{x}8} \frac{1}{1+\frac{1}{2}} = 667$ pounds per lineal foot, and $Ma = 1000 \times \frac{8^2}{24} \frac{1}{1+\frac{1}{2}} = 1780$ foot-pounds, or the moment at $C = -667 \times 8 + 1780 = -3553$ foot-pounds, and in the center of the span at $D = 1000 \times \frac{8^2}{8} - 3553 = 4447$ foot-pounds. Assuming the earth pressure $= 360$ pounds per lineal foot, $T = 360 \times \frac{4}{5}\text{ x } \frac{1+\frac{7}{16}}{1+\frac{1}{2}} = 276$ pounds per lineal foot, and $Ma = 360 \times \frac{4}{15}\text{ x } 8\,\frac{1+\frac{15}{80}}{1+\frac{1}{2}} = 610$ foot-pounds,, the moment at $C = -276 \times 8 + 610 - 360 \times 5 = -3398$ foot-pounds approximately, and moment at $D = 3398$ foot-pounds; hence the maximum moment at $C = -3553 - 3398 = -6951$ foot-pounds.

The favorable influence of the earth pressure at D should not be taken as the full 3398 foot-pounds, but ½ of it is certainly safe; and the moment at $D = 3447 - 1694 = 2753$ foot-pounds. Assuming a thickness of wall and slab of 9″ the reinforcement at D should be ¼% and at C=¾%.

REINFORCED CONCRETE CHIMNEYS.

The wind pressure assumed in the calculation of the chimneys in the tables=30 pounds per square foot of projected area of the chimney, which is equivalent to about 50 pounds per square foot on a flat surface. Let d=the outside diameter in feet and H=the height of the chimney in feet the moment about the base of the chimney=$15\ d\ H^2$ foot-pounds.

The moment of resistance of a ring section, reinforced by p per cent of steel is very closely=

$$\frac{3.14}{4}\ D^2t\ (1-3\frac{t}{D})+\frac{3.14}{4}\ Dptx15=$$

$$\frac{3.14}{4}\ D^2\ t\ \left\{ 1-3\frac{t}{D}+15p \right\}\ \text{in inch}^3,$$

when D outside diameter in inches,

and t thickness of ring in inches.

The stresses from the bending moment plus the direct compressive stresses from the weight of the chimney should not exceed 500 pounds per square inch. We assumed that the concrete does not take up any tensile stresses, and that the steel must take care of all tensile stresses.

The size of the footing is governed by the condition that there should exist only compressive stresses on the base (or as it is commonly said, that the resultant of weight and wind pressure intersects the base within the middle third, and to accomplish this it is necessary to place the footing at certain depth below the ground in order to obtain the benefit of the weight of the fill above the base. The lower portion of the chimney is generally provided with an inside shell of concrete, brick or fire-brick, 4″ thick, and an air space of 4″ is provided between the two shells. This is the reason why the lower shell has an inside diameter 16″ larger than the upper shell. We assumed the height of the inner shell 40′ for chimneys 5′ in diameter, 50′ for chimneys 6′ in diameter, and 60′ for all other chimneys.

REINFORCED CONCRETE CHIMNEYS.

Inside Diameter	5′–0″					
Height in feet	80	90	100	110	120	130
Thickness of lower shell	8″	8	8	8	8	8
Thickness of upper shell	6″	6	6	6	6	6
Percentage of Reinforcement in Lower Shell — Bottom	.75.	1.0	1.2	1.4	1.6	2.
Percentage of Reinforcement in Lower Shell — Top	.35	.35	.35	.50	.75	.95
Height of Minim. Reinforcement in Upper Shell	44′	44	44	44	44	44
Side of Footing—Feet	15′	18	18	20	20	22
Depth of Footing—Inches	30″	32	34	37	40	41
T	5′	4	4–6	5–0	5–6	5–6
Total Steel—Pounds	5600	7090	8470	11480	13800	16280
Total Concrete—Cubic feet	1270	1590	1740	2080	2230	2510

Inside Diameter	6′–0″						
Height in feet	90	100	110	120	130	140	150
Thickness of lower shell	8	8	8	8	8	8	10
Thickness of upper shell	6	6	6	6	6	6	6
Percentage of Reinforcement in Lower Shell — Bottom	.75	1.00	1.20	1.40	1.60	2.00	1.50
Percentage of Reinforcement in Lower Shell — Top	.35	.35	.40	.50	.80	.80	.95
Height of Minimum Reinforcement in Upper Shell	50	50	50	50	50	50	50
Side of Footing—Feet	17	18	20	20	22	24	25
Depth of Footing—Inches	34	38	39	43	44	45	48
T	4–6	5–0	5–0	6–6	6–0	5–6	6–0
Total Steel—Pounds	7330	8930	11060	13020	16030	19200	21000
Total Concrete—Cubic feet	1730	1985	2120	2500	2805	3140	3600

Inside Diameter	7′–0″						
Height in feet	100	110	120	130	140	150	160
Thickness of lower shell	8	8	8	8	8	10	10
Thickness of upper shell	6	6	6	6	6	6	6
Percentage of Reinforcement in Lower Shell — Bottom	.75	1.20	1.25	1.40	1.70	1.25	1.75
Percentage of Reinforcement in Lower Shell — Top	.35	.35	.40	.60	.75	.75	.90
Height of Minimum Reinforcement in Upper Shell	52	52	52	52	52	52	52
Side of Footing—Feet	20	20	22	24	24	25	26
Depth of Footing—Inches	38	41	43	45	48	50	53
T	5–0	6–6	6–0	5–0	6–6	6–0	6–0
Total Steel—Pounds	9120	12440	14380	16600	20150	21620	27000
Total Concrete—Cubic feet	2300	2560	2950	3260	3415	4070	4450

REINFORCED CONCRETE CHIMNEYS.

Inside Diameter	8'-0"					
Height in feet	110	120	130	140	150	160
Thickness of lower shell	8"	8	8	8	10	10
Thickness of upper shell	6"	6	6	6	6	6
Percentage of Reinforcement in Lower Shell Bottom	1.00	1.1	1.3	1.5	1.25	1.50
Top	.35	.35	.45	.70	.65	.75
Height of Minim. Reinforcement in Upper Shell	59'	59	59	59	59	59
Side of Footing—Feet	20'	22	24	24	25	26
Depth of Footing—Inches	43"	44	46	50	52	54
T	5'-6"	5-6	5-0	6-0	5-6	6-6
Total Steel—Pounds	12820	14440	16850	20180	25850	30250
Total Concrete—Cubic feet	2810	3160	3525	3780	4785	5125

Inside Diameter	9'-0"						
Height in feet	120	130	140	150	160	170	180
Thickness of lower shell	8"	8	8	8	10	10	10
Thickness of upper shell	6"	6	6	6	6	6	6
Percentage of Reinforcement in Lower Shell Bottom	1.0	1.1	1.40	2.0	1.2	1.5	2.0
Top	.35	.35	.55	.65	.60	.80	.95
Height of Minimum Reinforcement in Upper Shell	62'	62	62	62	62	62	62
Side of Footing—Feet	22'	24	25	26	27	28	28
Depth of Footing—Inches	46"	48	50	52	55	57	61
T	6'-0"	6-6	6-0	6-6	6-6	6-0	7-0
Total Steel—Pounds	4830	16760	20530	25820	25700	30570	36600
Total Concrete—Cubic feet	3415	3820	4145	4490	5250	5635	6000

Inside Diameter	10'-0"						
Height in feet	130	140	150	160	170	180	190
Thickness of lower shell	8"	8	8	10	10	10	10
Thickness of upper shell	6"	6	6	6	6	6	6
Percentage of Reinforcement in Lower Shell Bottom	1.	1.20	1.40	1.1	1.20	1.5	2.0
Top	.35	.45	.60	.60	.70	.80	.90
Height of Minimum Reinforcement in Upper Shell	67'	67	67	67	67	67	67
Side of Footing—Feet	24'	25	26	27	28	29	30
Depth of Footing—Inches	50"	51	54	57	60	62	64
T	5'-6"	5-6	6-0	6-0	6-0	6-6	7-0
Total Steel—Pounds	17500	20300	23850	27200	30150	33450	40850
Total Concrete—Cubic feet	4120	4450	4825	5800	6175	6525	6975

REINFORCED CONCRETE CHIMNEYS.

Inside Diameter	11'–0"						
Height in feet	140	150	160	170	180	190	200
Thickness of lower shell	8"	8	8	10	10	10	12
Thickness of upper shell	6"	6	6	6	6	6	6
Percentage of Reinforcement of lower shell Bottom	1.0	1.2	1.6	1.1	1.4	1.8	1.4
Top	.40	.50	.65	.65	.75	.80	.75
Height of Minim. Reinforcement in Upper Shell	73'	73	73	73	73	73	73
Side of Footing—Feet	25'	26	27	28	29	30	31
Depth of Footing—Inches	52"	55	57	60	62	64	67
T	5'–6"	7 –0	7 –0	6 –0	6 –6	6 –6	6 –6
Total Steel—Pounds	20550	24250	33350	29950	36450	43900	40250
Total Concrete—Cubic feet	4725	5175	5575	6450	6925	7350	8125

Inside Diameter	12'–0"						
Height in feet	150	160	170	180	190	200	225
Thickness of lower shell	8"	8	8	10	10	10	15
Thickness of upper shell	6"	6	6	6	6	6	8
Percentage of Reinforcement of lower shell Bottom	1.1	1.2	2.2	1.1	1.5	2.	.9
Top	.45	.60	.75	.65	.75	.90	.9
Height of Minim. Reinforcement in Upper Shell	76'	76	76	76	76	76	90
Side of Footing—Feet	27'	28	29	30	31	32	33
Depth of Footing—Inches	57"	60	62	65	67	69	77
T	6'–6"	6 –6	6 –6	6 –6	6 –6	6 –6	6 –0
Total Steel—Pounds	24400	27700	37450	34550	40800	48400	64000
Total Concrete—Cubic feet	5650	6100	6575	7525	8050	8600	11400

Inside Diameter	13'–0"					
Height in feet	160	170	180	190	200	225
Thickness of lower shell	8"	8	10	10	10	15
Thickness of upper shell	6"	6	6	6	6	8
Percentage of Reinforcement of lower shell Bottom	1.2	1.8	1.0	1.25	1.50	1.00
Top	.55	.65	.60	.75	.80	.85
Height of Minim. Reinforcement in Upper Shell	80'	80	80	80	80	94
Side of Footing—Feet	28'	29	30	31	32	34
Depth of Footing—Inches	63"	65	68	71	74	80
T	7'–0"	7 –6	7 –0	7 –0	6 –6	6 –0
Total Steel—Pounds	28900	35900	35250	40750	45800	65800
Total Concrete—Cubic feet	6550	7000	8075	8625	9200	13100

REINFORCED CONCRETE CHIMNEYS.

Inside Diameter	14'–0"					
Height in feet	170	180	190	200	225	250
Thickness of lower shell	8"	10	10	10	12	15
Thickness of upper shell	6"	6	6	6	8	10
Percentage of Reinforcement of lower shell Bottom	1.40	.9	1.0	1.25	1.40	1.65
Top	.60	.55	.65	.75	1.00	.85
Height of Minim. Reinforcement in Upper Shell	84'				98	115
Side of Footing—Feet	29'	30	31	32	34	35
Depth of Footing—Inches	67"	70	72	73	80	87
T	8'–0"	7 –6	7 –6	7 –6	7 –6	5 –6
Total Steel—Pounds	35350	37400	41700	47400	67700	104700
Total Concrete—Cubic feet	7425	8450	9100	9550	12200	17350

Inside Diameter	15'–0"					
Height in feet	180	190	200	225	250	275
Thickness of lower shell	8"	10	10	12	15	18
Thickness of upper shell	6"	6	6	8	10	12
Percentage of Reinforcement of lower shell Bottom	1.8	.90	1.0	1.25	1.2	1.60
Top	.65	.60	.65	.85	.70	1.00
Height of Minim. Reinforcement in Upper Shell	88'			106	120	137
Side of Footing—Feet	30'	32	33	34	35	37
Depth of Footing—Inches	70"	72	74	81	89	96
T	8'–0"	7 –0	7 –0	7 –6	7 –6	5 –0
Total Steel—Pounds	43950	42000	46500	62900	77900	140100
Total Concrete—Cubic feet	8275	9600	10200	12800	15950	23250

Inside Diameter	16'–0"					
Height in feet	200	225	250	275		
Thickness of lower shell	10"	12	15	18		
Thickness of upper shell	6"	8	10	12		
Percentage of Reinforcement of lower shell Bottom	1.0	1.00	1.10	1.50		
Top	.60	.65	.70	.75		
Height of Minim. Reinforcement in Upper Shell	92'	110	129	145		
Side of Footing—Feet	34'	35	36	36		
Depth of Footing—Inches	76"	84	91	100		
T	6'–6"	6 –0	5 –0	5 –0		
Total Steel—Pounds	48300	66800	99800	140000		
Total Concrete—Cubic feet	11300	14750	19800	24700		

REINFORCED CONCRETE CHIMNEYS.

Inside Diameter	18′–0″				
Height in feet	225	250	275	300	
Thickness of lower shell	10″	12	18	18	
Thickness of upper shell	8″	10	12	12	
Percentage of Reinforcement of lower shell Bottom	1.20	1.7	1.00	1.80	
Top	.70	.80	.35	.90	
Height of Minim. Reinforcement in Upper Shell	124′	145	190	190	
Side of Footing—Feet	35′	36	36	40	
Depth of Footing—Inches	87″	96	100	104	
T	8′–6″	7–0	6–0	9–0	
Total Steel—Pounds	67600	110500	122000	172800	
Total Concrete—Cubic feet	14700	19950	25800	29400	

Inside Diameter	20′–0″				
Height in feet	225	250	275	300	350
Thickness of lower shell	10″	12	15	18	24
Thickness of upper shell	8″	10	10	12	15
Percentage of Reinforcement of lower shell Bottom	1.20	1.20	1.20	1.20	1.00
Top	.60	.70	1.00	.85	.40
Height of Minim. Reinforcement in Upper Shell	129′	154	154	190	*
Side of Footing—Feet	36′	37	38	40	45
Depth of Footing—Inches	87″	96	100	108	114
T	7′–6″	6–6	6–6	6–0	6–0
Total Steel—Pounds	74400	105200	135500	153300	185200
Total Concrete—Cubic feet	16200	21800	25150	29950	37800

*The thickness of the upper shell is reduced in two steps to 6″.

The tables gives the thickness of the lower and upper shell in inches, the percentage of longitudinal reinforcement at the base and at the top of lower shell, which top reinforcement should extend into the upper shell. As minimum longitudinal reinforcement we assumed 0.35%, and the figures in the sixth line give the height from the top of the chimney down to which this minimum reinforcement may be used. The side of the square footing and its depth at the center is given in the next two lines, and the depth of the footing below the ground is given in the next line. The sectional area of the bottom reinforcement in the footing is given by 0.015 times the side of footing in feet by depth of footing in inches; part of which reinforcement may be obtained by bending the steel rods from the lower shell into the lower part of the footing. The reinforcement in the upper portion of the footing may be assumed as half that of the lower reinforcement. The volume of the concrete in cubic feet in the footings is approximately=1/20 times square of side in feet times depth in inches. The quantities of concrete and steel in the entire chimney are given in the last two lines. The concrete mixture in the footings should be 1:6, in the shells of the chimney 1:5. A wet mixture of concrete and small gravel, rather than crushed rock, should be used for the shells.

For example: A chimney 13′ 0″ inside diameter and 180′ high, has a lower shell of 10″ thickness and an upper shell of 6″ thickness. The reinforcement at the bottom of the lower shell is 1%, or $10'' \times 12'' \times \frac{1}{100} = 1.2$ square inches per lineal foot circumference, or 1″ round bars 7½″ c. c. The reinforcement at the top of the lower shell is 0.6%, or 0.72 square inches per lineal foot, or 1″ rods 13″ c. c. This latter reinforcement projects into the upper shell. The reinforcement at the top of the upper shell $= 0.35\% = 6 \times 12 \frac{0.35}{100} = 0.25$ square inches or ⅝″ round bars 14″ c. c. This minimum reinforcement may be adopted for a height of 80′ feet down, and then must be gradually increased till it reaches the value of 0.72 square inches at the junction of both shells.

The lap of the rods should not be less than 50 diameters and they should be staggered. The base is 30′ square and 68″ deep at the center and can be made 12″ deep at the edges. The reinforcement in each of the two main directions =0.015×30×68=30.5 square inches, or about thirty-nine 1″ round bars in each direction. The base of the footing must be 7′ below the ground, and in the entire chimney are 807 cubic feet of concrete and 35250 pounds of steel.

The circular reinforcement of the chimneys was assumed =¼%, or 0.24 and 0.18 square inches per lineal foot in the 8″ and 6″ shells, respectively.

The compression per square foot on the ground was assumed less than twice of the weight of dirt above the bottom of the base.

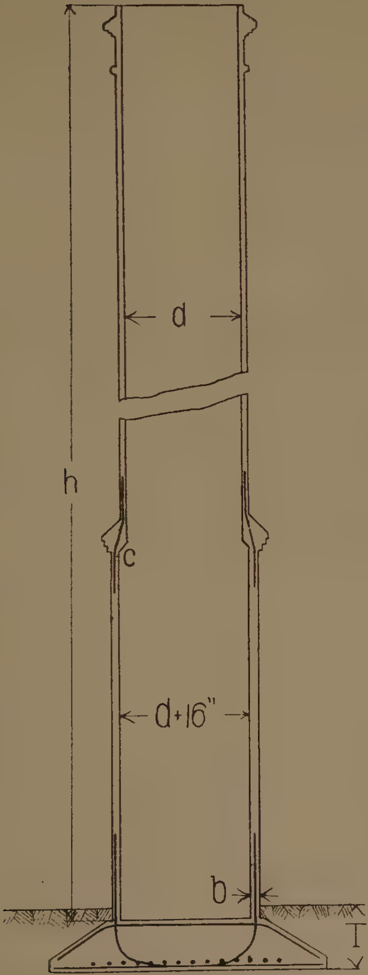


Fig. 88. Typical Reinforced Concrete Chimney.

TROLLEY AND TRANSMISSION POLES.

Reinforced concrete poles deserve more serious consideration on the part of users of poles than is at present given to them. The cost of replacing a wooden pole is generally higher than the original cost of concrete poles.

An ordinary wooden pole, as used for telephone and telegraph wires, ordinarily fails under a pull not much above 1000 pounds at the cross-arm, either by breaking or more generally by pulling out of the ground.

We recommend a factor of safety of two, or maximum three, for reinforced concrete poles.

Example: On a pole is exerted, at a height of 20′ above the ground, a force of 1000 pounds. The moment at the ground line $= 1000 \times 20 \times 12 = 240{,}000$ foot-pounds. Experiments made by the writer and others indicate that the ultimate bending moment $= 2000 \times$ moment of resistance. For a factor of safety of two we require for above pole a moment of resistance $= 2 \times 240{,}000 \div 2000 = 240$, or, according to table on page 205, a pole of a side of 11″ at the ground line reinforced by four ⅝″ bars will suffice.

The side of the pole at the top should be made about ½ of that at the base. The writer's experiments demonstrate that the deflection of poles, when figured for a factor of safety of two $= 0.5 \frac{L^3}{I}$ inches, when L the leverage in feet and I the moment of inertia in inch^4, and for a pull of 1000 pounds. In our case the deflection $= 0.5 \times 8000 \div 1335 = 3.0$ inches. This pole contains, if we assume a length of 27 feet, 15.1 cubic feet of concrete and 113 pounds of steel.

The anchorage is considerably increased by imbedding in the ground short pieces of concrete beams about 15″ to 18″ below the ground line. For important lines it will always pay to test the anchorage for the various soils encountered.

TROLLEY AND TRANSMISSION POLES.

Side of Square Pole	18"		16"		15"		14"		13"		Weight of Four Rods in Pounds
I of Plain Section	8750		5450		4225		3190		2380		
Reinforcement 4–⅝ Rounds	9850	**1090**	6285	**785**	4940	**658**	3790	**530**	2870	**440**	4.2
4–¾ Rounds	10330	**1140**	6660	**832**	5315	**710**	4060	**580**	3110	**480**	6.0
4–⅞ Rounds	10900	**1210**	7100	**889**	5635	**750**	4370	**624**	3370	**520**	8.2
4–1" Rounds	11570	**1280**	7600	**950**	6065	**810**	4740	**678**	3680	**570**	10.7
4–1⅛ Rounds	12310	**1370**	8160	**1020**	6515	**875**	5150	**735**	4030	**620**	13.5
4–1¼ Rounds	13150	**1416**	8800	**1100**	7105	**950**	5610	**802**	4380	**675**	16.7

Side of Square Pole	12"		11"		10"		9"		8"		7"		Weight of Four Rods in Pounds
I of Plain Section	1728		1010		833		549		342		200		
Reinforcement 4–⅝ Rouuds	2138	**355**	1335	**242**	997	**200**	671	**150**	429	**107**	258	**74**	4.2
4–¾ Rounds	2328	**390**	1485	**270**	1088	**217**	742	**166**	480	**120**	292	**83**	6.0
4–⅞ Rounds	2543	**425**	1660	**300**	1203	**240**	827	**184**	540	**153**	333	**95**	8.2
4–1" Rounds	2798	**468**	1860	**338**	1343	**268**	929	**206**	612	**135**	381	**109**	10.7
4–1⅛ Rounds	3088	**515**	2090	**380**	1493	**298**	1044	**234**	697	**174**	437	**125**	13.5
4–1¼ Rounds	3398	**565**											16.7

The black figures denote the moment of resistance in inch^3. The figures to the left, the moment of inertia in inch^4.

LABOR COST OF REINFORCED CONCRETE WORK.

The cost of material, delivered at the site, can be obtained without great difficulty.

A 1:6 concrete mixture requires per yard of finished concrete 1½ barrels of cement, 0.9 yards of rock and 0.45 yards of sand. A 1:7½ mixture requires 1¼ barrels of cement and 0.95 and 0.48 yards of rock and sand, respectively; and a 1:9 mixture requires one barrel cement and one yard of rock and ½ yard of sand.

The prices of the materials must be figured unloaded at the mixer. Unloading from railroad cars at the mixer costs generally two to three cents per barrel of cement, 15 cents per yard of rock and 10 cents per yard of sand.

The labor hours per cubic yard of concrete for mixing, hoisting, wheeling and tamping varies from 1⅓ for a well-proven, well-designed new plant, using bins or steel carts for filling the mixer, an elevated water tank, first-class hoisting bucket, well-designed hoppers, and carts or steel cars for transporting the concrete, to two hours for an old plant with less favorable conditions of loading the mixer.

To obtain this low cost, at least 45 batches an hour must be run through a mixer of nominal ¾ yard capacity, which, however, does not make more than 10 cubic feet of concrete for a 1:6 mixture, when a batch consists of two bags of cement, 4 cubic feet of sand and 8 cubic feet of rock.

For the average plant it requires 4 labor hours for each cubic yard of concrete, including average stoppages on account of breakdowns, etc. Only in exceptional cases it takes six to eight labor hours per yard of concrete, even where hand mixing is resorted to, and where the concrete is hoisted by crude means.

The installation of the plant (not including the cost of mixer, hoisting engine, etc.) cost from 20 to 60 cents per yard.

Steel Labor: The average labor hours per ton of steel is 24, and should in no case exceed 36. Under very favorable conditions they may be as low as 10. Under good management three men will bend 30 to 35 high-carbon bars into a hog chain form in one hour, on a well-built bench, using two extra heavy gas-pipes for a lever.

One man can make at least 200 stirrups, as shown in Figure 2, in one hour.

Carpenter Work: The taking down of the forms and the removing of the lumber from the floor takes from 10 to 20 labor hours per 1000 feet B. M.

For the erection of the forms it takes, under the best management, not less than 20 carpenter and 10 labor hours per 1000 feet B. M., and these hours may increase in a badly designed structure and under correspondingly bad management to 80 carpenter hours and 20 labor hours.

Brackets and a great variety of beams and spacing of beams increase the form labor very rapidly, as well as high ceilings.

Nails: It takes about 6 to 8 pounds of nails per 1000 feet B. M.

After the rough work is completed, the patching, cleaning up, the finishing of floors, stairs, walls and of sundry minor details, which were omitted when building the main structure, the carting away of all debris, lumber, dismantling of plant, etc., cost often from 10 to 50% of that of the main work.

Water: It requires in the average 80 to 150 gallons of water per cubic yard of concrete, which includes the water for mixing, wetting of the forms, sprinkling the concrete, etc. In many cities a charge of 5 to 10c per cubic yard of concrete is made for the water.

GENERAL SPECIFICATIONS FOR REINFORCED CONCRETE WORK.

The contractor shall have at least three years' experience in reinforced concrete construction, and shall mention in his propositions five of the most important reinforced concrete structures which were built under his direction. If he expects to give the direction of the work to one of his employees, he shall bring verified testimonials that his foreman has at least three years' experience in this line.

The execution of the work will be supervised, on behalf of the owner, by a competent engineer, having at least three years' experience in reinforced concrete and ten years' experience in building construction, and his decisions in all doubtful points must be followed. If the contractor is convinced that the decisions of the engineer are wrong and would cause him serious injury, he shall have the right to ask from the engineer a written statement of his decision and its causes, and to have samples and photos certified, and if it should be decided by a competent board of arbitration that the engineer was wrong, the contractor will be paid the actual extra cost plus 15%.

MATERIALS.

The cement shall meet the requirements of the specifications of the American Society of Civil Engineers.

One bag containing 94 pounds of cement shall be assumed=1 cubic foot.

At least one week's supply of cement shall be stored in a shed at the building site, and each carload of cement shall be accompanied by a certified test from the mill, stating the probable age of the cement, and shall also be tested by a competent laboratory selected by the owner.

AGGREGATES.

As sand, shall be classified all materials, either natural sand or finely crushed rock passing a sieve of ¼″ holes. The sand shall be obtained from (state pits), and shall be free from vegetable and other deleterious matters, especially lumps of clay.

The use of sand having grains of 1/64″ diameter or smaller, to the extent of 50%, is absolutely prohibited. Where there is a possibility of choice, a sand containing about 30% of fine material and 70% of material about four times coarser is best to use, if the expense of obtaining such a sand is not prohibitive.

The crushed rock or gravel shall be clean, hard, durable and free from all deleterious matters, especially lumps of clay.

The particles shall pass a one-inch ring for all column girders, floor slabs less than 5″ thick and walls less than 8″ thick; and shall pass a two-inch ring for ordinary footings, thick floor slabs and walls less than 3′ thick. For very heavy foundations and walls, stones up to 6″ diameter, or sometimes even larger stones, may be imbedded to the extent of 50% of the aggregate.

The presence of 2% (in weight) of finely divided clay in the aggregate shall be permitted; a larger percentage will be rejected.

The stone need not be screened, if the percentage of stone screenings does not exceed ten. Also here about 30% of finer material and 70% of material four times larger gives the best results.

WATER.

The water shall be free from oil, acids, strong alkalies, or other deleterious matters. The contractor shall provide an ample system of water distribution, in order to facilitate the cleaning of forms, and the sprinkling of new concrete.

STEEL.

The steel shall conform to the manufacturer's standard specifications. High-carbon steel shall have an ultimate strength of 90,000 pounds per square inch and a yield point of at least 50,000 pounds. These rods shall bend cold around a radius equal three times their diameter to an agle of 90° without breaking. Cold twisted steel rods shall have a yield point of 50,000 pounds and an ultimate strength of about 85,000 pounds per square inch.

Should, in case of contingency, the contractor be compelled to use mild steel rods in lieu of high-carbon bars, he shall increase the sectional area in slabs by 10% and in girders by 20%; in columns, either mild steel or high-carbon rods may be used.

MIXING AND PLACING OF CONCRETE

The concrete shall be mixed in the proportion of one cubic foot of cement to about six feet of aggregates, sand and stone being measured separately. As a rule, the proportion of sand to stone shall be 1:2, and the exact proportion shall be found before starting concreting by making a small test beam. A good concrete worker can tell whether the concrete allows of easy tamping and runs freely around the steel rods. Should there be any dispute in regard to the quantities of aggregate to be used for each bag of cement, it shall be decided by a test, and the meaning of this specification is fulfilled when 1.45 barrels of cement are contained per cubic yard of finished concrete. If the engineer insists on more cement, the contractor shall be paid cost and 10% extra.

The materials shall be measured in a uniform manner, in wheelbarrows or otherwise, so that the quantities can be easily controlled by the workmen and engineer.

The mixing shall be done in a concrete batch mixer of sufficient capacity, and the materials, including water, shall turn in the mixer not less than ½ minute. Hand mixing on a tight platform shall only be resorted to for unimportant work, and the materials shall be turned over at least four times until they are homogeneous in color and consistency. The concrete should be mixed wet enough to flow freely around the reinforcement and to fill out all the corners in the forms. Great care is to be taken that there is not too much water used, thereby causing a separation of motar and of the coarse aggregate. Should a less amount of water give still unsatisfactory results, the proportion of sand to rock must be changed.

No concrete shall be used which has partially set (the setting time is known from the cement tests), and no retempering shall be permitted.

Before placing the concrete, the forms must be well cleaned and thoroughly wetted, and wood shavings must be removed from the floor, in order to prevent the wind from blowing them into the forms.

No concreting of columns, beams or walls shall be commenced unless the forms are previously inspected, by the engineer, for cleanliness and straightness, and the proper location of the reinforcement.

In using wet concrete a great deal of laitance is formed on top of the concrete, especially in columns, girders and walls, which must be removed before starting concreting after a suspension. The concreting shall be stopped in girders and slabs in the center, unless prevented by the inclemency of weather, or similar reasons, in walls preferably in steps. When starting up concreting after a suspension of only a day, the joints should be thoroughly cleaned, and drenched with water, and covered with grout. For a longer suspension, remove about ¼ to 2 inches of concrete at the joint.

It is preferred that the columns are concreted one day ahead of the girders, and the girders a few hours ahead of the floor slabs.

At least one man shall continuously tamp a column, during process of concreting, by means of a stick of lumber 1″x2″ or 2″x2″. At least one man, but generally two men, will be required to tamp each girder, during the process of concreting, by means of special spades, and the contractor shall supply a sufficient number of crowbars and heavy hooks for shaking and eventually lifting of the reinforcement. Care shall be taken that the stirrups remain in their places and are lifted up after the form is filled.

Care shall be taken that the floors are brought up to the right level, and a levelling instrument shall be always on the floor, ready for checking.

Where the floors are to be cement finished, as for warehouses, factories, etc., the rough concrete floor shall be levelled to a level ½″ below the top of the finished floor. As an absolutely level floor in rough (or even finished concrete) concrete cannot be obtained, this will give a cement finish averaging ¾″ thick.

The cement finish, if figured as carrying stress, shall not be placed later than two days after the rough concrete is placed, and care shall be taken that the rough concrete is thoroughly cleaned and brushed with a steel brush and drenched with water before applying the finish. For office buildings, hotels, etc., the cement finish may be applied after several months, but shall not be less in thickness than 1″, and the rough concrete floor shall be washed with a 20% solution of commercial muriatic acid and thoroughly drenched with water before applying the finish. The cement floor shall be covered with sand and kept wet for several days. No walking on this floor shall be allowed for two days, unless boards are spread over it.

Where the finish is applied at once, and where concreting is done on the story above, the cement drippings must be thoroughly washed off and cleaned off every day, in order to preserve a smooth floor.

The cement finish shall consist of one part of cement to two parts of coarse sand, or granite screenings. After the forms are taken out all places showing honeycombs or other imperfections shall be neatly patched up.

The faces of concrete exposed to the sun and wind must be drenched with water for at least three days, and occasionally sprinkled for a week or more, or even covered with a layer of wet sand in case of cement finished floors.

Where concrete is to be deposited in water, it shall be done by means of a tremis or bottom dumping bucket, and divers shall be occasionally sent down to remove laitance and to inspect the concrete.

FREEZING WEATHER.

When the temperature is below freezing and above 20°, the concreting may be carried on by using boiling water for mixing (tests shall, however, be made by the laboratory whether the hot water does not injure that particular brand of cement, in regard to strength), and by pouring boiling water on the forms, and especially the reinforcement, so that not a particle of ice or snow remains. The concrete must be immediately covered by planks and tarpaulins. No concreting in cool weather shall be commenced unless all preparations for boiling water are made and the required quantity of tarpaulins are at the site.

FORM WORK.

The form work must be carefully designed for strength and deflection, and in no case should less material be used than given in this Hand Book. The forms shall be erected as true as practicable, and no variation of more than ¼″ shall be allowed (if another variation is desired, it must be considered that the cost of the form work increases very rapidly for a greater precision).

The forms shall be built fairly tight, to prevent a gross leakage of motar, producing honeycombs and loss of strength of the concrete. The leakage of water, colored by cement, will not be considered injurious. The floor boards shall be surfaced on one side (S. 1 S.); the side and bottom of the beam forms shall be S. 1 S., or surfaced one side and one edge (S. 1 S. & 1 E), according to the purpos for which the structure is intended.

The column boards shall be surfaced one side and both edges (S. 1 S. & 2 E.), in order to facilitate the plumbing of the forms. Right-angle triangular strips (1″ or 2″ sides) shall be placed in all corners of column and beam forms, and the beam forms shall be given a camber of ½″ or 1/300 of the span.

Column and wall forms may be removed in three days. The removing of beam and slab forms depends on the ratio of total load to dead load for which they are designed, and on the weather conditions. The proper time can be found by making small test beams, at the same time and of the same concrete of which the structure is built. Lacking these tests, the supports of the beams shall not be removed before three weeks in warm, or four weeks in cool weather. The sides of the beams may be removed at the same time as the slab forms.

If the slabs are of short spans, say not exceeding 8′, the forms may be removed in five days in warm weather and eight and fourteen days in cool weather. For larger spans the floor boards may be removed at the same time, but temporary props must be placed in the center of the panels and must remain for at least two weeks, and for a longer time if the floor supports the forms for the story above.

THE PLACING OF THE REINFORCEMENT.

Before the reinforcement is placed, the forms must be cleaned of all dirt and shavings and care must be taken to prevent new dirt and shavings from coming into the forms. In case of wall and column footings, a 2″, or sometimes 4″, layer of concrete shall be spread on the ground before the rods are placed. Rods in wall footings do not need to be wired; in column footings, however, each rod must be wired at two points by No. 16 wire. The column rods should be rigidly wired to the coils, so that they retain their desired spacing in handling and concreting. The column reinforcement must be placed plumb and centrical and braced in two directions, and grout (1:2) must be poured into the footing holes or gas-pipe sleeves for proper connections. Care must be taken that the column rods keep their place while concreting the columns, and for this purpose the tops of the rods must be steadied by wires or boards.

In bending the girder and beam rods, care must be taken that the depth of the bent rods is not greater than given in the drawings, as otherwise the rods might project above the concrete floor.

The stirrups shall first be placed in all beam and girder forms as shown in Figure 3, then the straight rods, and afterwards the bent rods. Care must be taken that the rods do not come nearer to the surface than 1″ (1½″ or 2″), and that they equally overlap at both ends.

The floor rods shall be placed after the girder rods, and shall be wired at two points to keep their place. It will be insisted that they are placed in their exact position during the wiring; a variation of 1″ shall be permitted if during concreting the rods are shifted. The contractor shall supply his men with a sufficient number of hooks, to be used to raise the rods ½″ (¾″) above the floor during concreting.

The extra rods over the beams or girders shall be placed immediately after the slab is concreted over the beams, and pressed into the concrete.

Horizontal wall rods shall be wired to vertical rods at least every four feet, and proper provision, by means of staples, or otherwise, shall be made to keep them in the proper distance from the face of the wall.

The steel rods shall be stored in a clean place and shall be properly sorted, and shall be free from dirt, ice or heavy scales of rust, when placed into the forms. A slight coat of rust shall not be considered objectionable.

TESTS.

The contractor agrees that he is fully informed as to the purpose for which the before-mentioned structure is to be used, as described in the plans and specifications, and being skilled in the construction of reinforced concrete work, fully approves of the plans, design and specifications, and guarantees that this work, done in accordance to these plans and specifications, will fulfill the requirements for which it is intended in every respect.

The contractor shall test, on demand, at least one girder, beam, or panel to twice the live load for which the particular floor is intended, six weeks after the concreting, and no undue deflections, cracks or other signs of weakness shall appear. Should any failure or defect appear in any part of this work, either during test or while in use during a period of one year after date of completion, the contractor will remove such defect and provide construction adequate for proper fulfillment of this service for which this structure is intended at his own sole cost and expense, and without cost to the owner.

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ADDENDA AND CORRECTIONS

On page 105, in tenth line from below, read ‘‘page 95’’ instead of ‘‘page 92’’.

On page 107, in fourteenth line from below, read ‘‘2″x4″.’’

On page 112, in the equation for the moment of the weight of the earth about the center of the base, ‘‘$\frac{1}{16}h$’’ should stand for ‘‘$\frac{1}{16}$’’.

On page 118, in the eleventh line from top, read: ‘‘The cost per square foot of shell’’

On page 133, the weight of steel per lineal foot of square bin should be doubled. In case of a cluster of bins, care must be taken to divide the unit quantities of concrete, steel and form lumber by two for all interior bins. For the outside walls the full quantities must be taken.

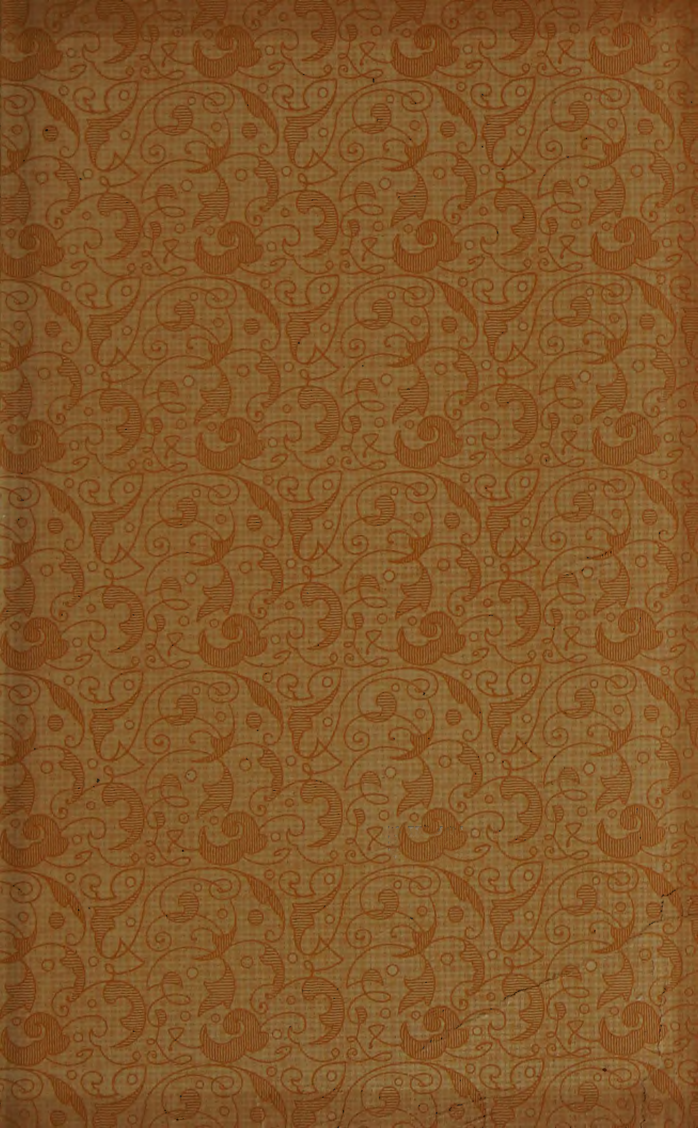
On page 153 the heading of the second column should have been ‘‘$T \div l^2$’’ and the figures under heading ‘‘0.4x’’, ‘‘0.3x’’, ‘‘0.2x’’ and ‘‘0.1x’’ should have been printed in black.

On page 161 should have been noted that it is of advantage in reinforced concrete piles to tie the rods together by coils of No. 5 wire, spaced 6″ c. c., which spacing should be diminished to about 2″ near the top of the pile.

In Fig. 84, page 190, the rise of the roof is not clearly marked as ‘‘h_1’’.

On page 195 in second part of first equation read D^2pt instead of Dpt.

On page 205, ‘‘Weight of Four Rods in Pounds’’ should read, ‘‘Weight of Four Rods in Pounds per Lineal Foot.’’



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